A LABORATORY INVESTIGATION

OF THE PROPERTIES

OF SOIL-AGGREGATE MIXTURES

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one hundred per cent. A moisture-density determination was made for each mixture, and a series of triaxial shear tests were performed on specimens from each mixture compacted to the maximum dry density at the optimum moisture content. Strength characteristics, gradation and the volume relationships of the aggregate, binder soil, water, and air in each mixture were correlated with the binder soil contents of the various mixtures.

This investigation led to the following conclusions. In a soil-aggregate mixture there is a certain binder soil content at which the highest maximum dry density will occur, and at which the optimum moisture content will be a minimum. The addition of small quantities of binder soil to an aggregate results in definite improvement in the strength characteristics of the mixture over the individual components. The addition of excessive amounts of binder soil to an aggregate results in a mixture of lower strength than either of the component materials alone. The addition of small quantities of binder soil to an aggregate results in a sharp drop in the angle of internal friction, apparently due to the coating of individual aggregate particles with cohesive material. Mere conformance to American Society for Testing Materials gradation specifications will not insure the stability of a soil-aggregate mixture.

CHAPTER I

INTRODUCTION

General.--In general terms, the process of soil stabilization may be defined as the process by which the physical properties of soil may be improved by either mechanical or chemical treatment to produce a material more suitable for the use intended.

Within the two broad categories of chemical and mechanical stabilization lie many varied techniques. In the realm of mechanical stabilization, compaction is perhaps most highly developed and a widely recognized means for improving soil properties.

As the principles of soil compaction have become understood, they have been quickly applied to the construction of earth structures such as roadways, dams and fills. Particularly in the field of highway construction have the benefits of stabilization by compaction been recognized.

In many cases, compaction alone does not provide sufficient improvement. Attention has therefore been turned to other methods of stabilization in an attempt to improve substandard materials so they might adequately support increasing traffic loads. One of the early methods of stabilization, other than compaction, involved the mixing of soils to arrive at a roadway material more satisfactory than either of the original materials. Although this method of stabilization has been used for many years, relatively little is known about the mechanisms involved. The purpose of this thesis is to report upon a scientific investigation of such a mixture. The nomenclature "soil-aggregate mixture" has been chosen for use in this discussion.

Soil properties.--When considering soils for structural uses, the physical properties with which the engineer is most concerned are:

Strength
Compressibility
Swelling and shrinking
Permeability
Plasticity

These properties may be varied in a number of ways, one of which is by mixing of two or more different soils. Such mixtures may retain some of the characteristics of the constituents, or perhaps may have properties quite different from those of the components.

For convenience, soils may be considered to be of two basic types. The very fine-grained soils (cohesive) exhibit characteristics which are largely dependent upon adsorbed water films and inter-molecular actions, while the coarser-grained soils (cohesionless) have characteristics determined mainly by grain shape and size. The more prominent characteristics of these two basic soil types are as follows:

Cohesive soils
- Cohesion
- High compressibility
- Relatively large changes in volume with changes in moisture content
- Low permeability
- Plasticity

Cohesionless soils
- High internal friction
- Particle interlock
- Low compressibility
- Little or no volume change with changes in moisture content
- High permeability
- Little or no plasticity

The basic hypothesis of stabilization by mixing is that by combining the two basic soil types, a material with intermediate properties may be obtained. Nearly all of the applications of this concept, however, are based on "rule of thumb" procedures. The lack of rationality can be attributed in part to an insufficient knowledge of the engineering properties of soils and soil mixtures, and to inadequate testing methods.

The insufficiency of information concerning soil mixtures is indicated by the many different specifications developed by the various agencies and organizations based on past performance. Nearly all of the performance information lacks complete supporting data and omits evaluation of other conditions.

Perhaps the most valuable contributions to our knowledge of soil mixtures have been provided by five investigations.

Early in the 1930's, R. R. Proctor conducted investigations concerning compaction which have provided a better understanding of the mechanism involved, and have led to the development of a procedure for the field control of compaction.(1)

The relationships between density and gradation were investigated as early as 1907 when Fuller and Thompson reported their findings with regard to proportioning concrete.(2)

The Bureau of Public Roads performed a series of investigations extending from late in the 1930's through 1946, which took the form of simulated accelerated road tests.(3, 4, 5) Although empirical in nature, the close control of conditions produced test results of considerably more value than the results of previous performance-type tests.

An investigation of the effects of plasticity reported by Deklotz indicated the close relationship between Atterberg limits and bearing capacity.(6)

The Highway Research Board sponsored research at Purdue University in 1945 on the characteristics of soil-aggregate mixtures as a whole.(7) Correlations were made of compaction, density, and the California bearing ratio characteristics of several types of mixtures.

An investigation of similar nature has been undertaken by the author and is reported herein. The questions the author's research were intended to explore are concerned with the variations in maximum density and optimum moisture content with various proportions of coarse and fine particles, the values of strength of various mixtures, and with the volume relationships of the soil, air, and water in the mixture. A secondary objective of this paper is to interpret the results of the laboratory findings in terms of bearing capacities for vehicular wheels.

CHAPTER II

THEORY

General concepts.--In considering the problem of attaining maximum strength in a soil mixture, there are two general avenues of approach that one may follow. Both of these approaches are based on the premise that maximum strength will occur simultaneously with maximum density.

One theory requires the attainment of a "perfect" gradation. A "perfect" gradation may be illustrated in this manner: A quantity of maximum sized particles are arranged in their most dense state. The next smaller sized grains are the maximum size that will fit within the interstices without increasing the volume of the original granular mass. Sufficient numbers of this next smaller size are present to occupy all of the interstices. Each succeeding smaller size is the maximum size and of the correct quantity to occupy the spaces remaining between the larger sized grains. Based on grains of some definable geometric shape, the grain-size distribution would follow some mathematical progression extending to infinity. At this limit, the soil will have become a solid mass.

In the second approach to attaining maximum density, cohesive and cohesionless soils are considered as separate entities. The cohesionless material is envisioned in its most dense state. All of the remaining void space is filled with the fine-grained cohesive material, reducing the voids in the total soil mass to a minimum.

Neither of these concepts are totally compatible with actual conditions when consideration is given to non-geometric particle shape, natural grain-size distribution, and the electro-chemical actions involved with the clay size particles. The concept most often used is a modification of the second approach. This involves a cohesionless aggregate of relatively large sized particles. With this aggregate is mixed a fine-grained cohesive soil, referred to as a "binder" soil because of its function in employing adhesive properties to bind the large particles together. This approach may be recognized as being similar to the macadam method of pavement construction.

Idealized mixtures.--In such a soil-aggregate mixture, a primary problem is determining the correct proportions of the components. The fine-grained materials used as binders have characteristically low dry densities; they have adequate bearing capacities when they contain limited moisture, but they are unstable when they contain excessive water. High capillarity and low permeability are other noteworthy characteristics. Conversely, granular or semi-granular materials may have high dry densities, are free draining, have low capillarity, and are relatively unaffected by variations in moisture content. In combining these two types of materials, it is assumed that the properties of the mixture will depend largely upon the proportions of the ingredients.

One idealized concept of the effect of adding increasing quantities of binder soil to an aggregate is shown by Fig. 1. The aggregate is compacted to a given density.(Fig. 1a) Point-to-point contact is established between the individual grains. In this condition, shearing strength of the materials is dependent upon the intergranular friction and interlocking between the grains. It has been established that the strength due to friction is a function of the intergranular pressure, and that interlocking is independent of these pressures. According to Berry, interlocking appears

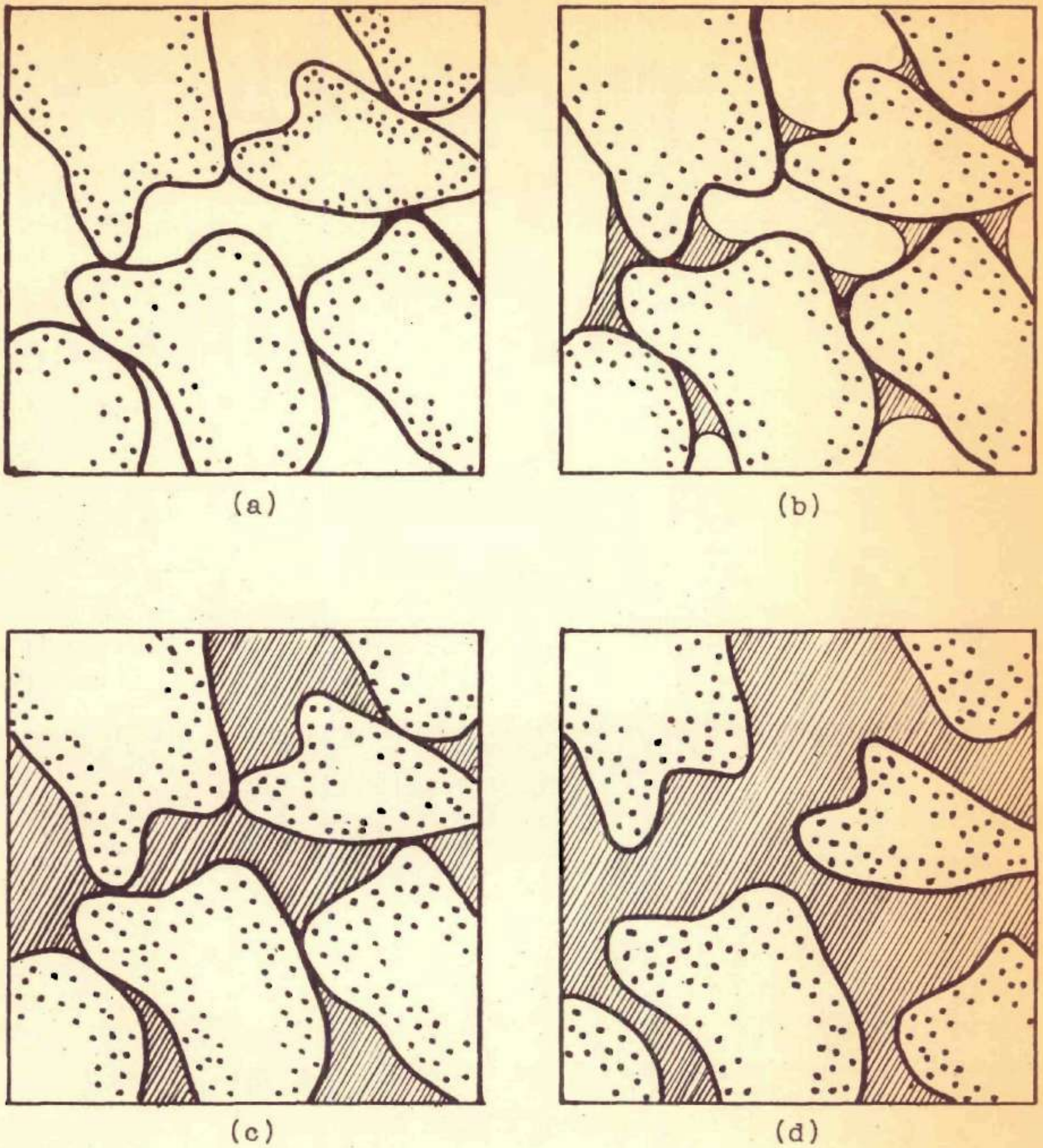


Fig. 1. Idealized Conditions Within a Soil-Aggregate Mixture

to be a function of grain shape.(8) Therefore, with any one material, strength due to interlocking remains a constant so long as the material remains at the same density.

If a small quantity of binder soil is added to the compacted aggregate, it is distributed throughout the void space between aggregate particles filling crevices and providing small amounts of cohesion to bind the particles together.(Fig. 1b) A slight increase in strength is probably realized as a result of the total cohesive force. If binder soil is added in larger quantities, a condition is reached where the soil binder exactly fills the voids between the aggregate particles.(Fig. 1c) At this point, a maximum cohesive force is acting, as well as both the intergranular friction and interlock of the aggregate particles, to resist shear of the mass.

The addition of any more binder soil to the mixture results in a condition where individual aggregate particles are floating in a matrix of the binder.(Fig. 1d) In such a condition, the strength of the mixture would be almost entirely dependent upon the properties of the binder, since the aggregate particles serve only as a filler and provide little, if any, frictional resistance.

The foregoing idealization envisions uniformly graded aggregate with much smaller binder soil particles. The concept probably remains the same, however, for well-graded materials, with the well-graded aggregates demonstrating greater strength because of increases in internal friction (due to a greater number of points of intergranular contact per unit volume).(9) The total cohesive force would probably be smaller, since the quantity of fine-grained particles in a given volume of the mixture would be smaller.

From this information it appears that a well-graded angular aggregate, in a dense condition, and with the voids filled with compacted cohesive binder soil, would produce a construction material of high density and strength. Difficulties arise, however, due to the tendency of some clays to swell or shrink with variation in moisture content. When increases in moisture content occur, the binder soil may increase in volume. This expansion forces separation of the aggregates, reducing the shearing resistance afforded by internal friction and interlocking. In this condition, the mixture has essentially the same strength as the binder alone.

Another consideration which should be made is that in combining an aggregate and a binder soil, it is virtually impossible to place the fine-grained material between grains of a previously compacted aggregate. The components must be combined in a loose state and then compacted. In so doing, it is highly probable that some of the cohesive binder soil will coat individual grains of aggregate, preventing full intergranular contact from the very outset. In addition to this, when binder soil is present in quantities less than that required to completely fill the voids, there is little possibility for compaction of the binder material.

Performance tests.--As was pointed out previously, almost all the information concerning soil-aggregate mixtures is in form of the roadway performance data. Although most of the information is notably incomplete, general conclusions have been drawn concerning the major influencing factors. The stability of soil-aggregate mixtures, which may be defined as its ability to resist excessive deformation, is apparently largely controlled by (a) the gradation of the aggregate, (b) the proportions of the component soils, (c) the plasticity of the fine-grained particles, and (d) the degree of compaction.

Following along each of the first three factors, variations in mix design methods have been developed. The fourth factor, compaction, has been recognized as an essential part in placing mixtures designed by any method.

Mix design by gradation.--The theoretical concept of "perfect" gradation is followed in attempting to arrive at a mix of maximum density with an assumed maximum strength. The earliest work of note in attaining maximum density through gradation was reported in 1907 by Fuller and Thompson following studies of concrete mixtures.(10) Mathematical curves were fitted to the grain-size distribution curves for the gradation which resulted in the highest strength concrete. These curves took the form of ellipses with straight line tangents.

For practical purposes, the gradation curves determined by Fuller and Thompson (later confirmed by Rothfuchs)(11) have been expressed in a general form referred to as the "Talbot equation":

$$P = 100\ (d/D)^n$$

Where d is a given grain size
P is the per cent finer than given grain size
D is the maximum size particle
n is a variable exponent

From the equation it is seen that, with a certain maximum size aggregate, variation in gradation may be obtained by varying the value of the exponent. When the exponent "n" equals 1.0, then the curve becomes a straight line; when the exponent "n" equals 0.5, the curve is a parabola. The foregoing expression has been used many times under the assumption that when the exponent "n" equals 0.5 it correctly defines an ideal gradation. In the work of Talbot and Richart, concrete aggregates were artifically graded to conform to the Talbot equation, using values for "n" ranging from 0.50 to 1.20.(12) Results indicated that the value of the exponent varied inversely as the maximum particle size. For maximum particle size of 0.186 inches (No. 4 sieve), a value of 1.10 for "n" gave maximum density. At the maximum size particle of two inches, the maximum density corresponded to a value of "n" equals 0.65.

Mix design by direct proportioning of the component parts.--Housel has advocated a method of proportioning a soil-aggregate mixture based on absolute volumes of the components.(13) In applying this method, absolute volume of the aggregate in a loose state is first determined. The remaining void space is then filled with soil binder proportioned on the basis of loose volume. The method is said to be adequate whenever the stabilization process depends primarily on water as a cohesive agent and plasticizer during compaction. This seems to indicate its applicability primarily to silty rather than clayey binder soils. In this instance, excessive quantities of binder soil would have less effect than if the binder were predominately clay.

In the direct proportioning of soils to meet predetermined specifications, graphical procedures seem to be widely employed. One method employs a triangular graph to determine the percentage of coarse and fine aggregates and binder soil to meet the specified gradation.(14, 15, 16) Control is possible only at three grain sizes, usually established as one or one and one-half inches, the No. 10 sieve and the No. 200 sieve. Another method may be used where only two materials are to be combined.(17, 18) This method also employs graphical procedures, however it permits control of any number of grain sizes.

Mix design on the basis of plasticity.--In a method developed by Hennes, the primary object is to arrive at a mixture with a predetermined plasticity.(19) The percentage of clay sized particles required to give the final mixture this plasticity is determined from the following equation:

$$C = \frac{c\ I}{i}$$

Where C is the required percentage of clay sized particles in the material finer than a No. 40 sieve
I is the specified plasticity index
c is the percentage of clay in the available binder soil
i is the plasticity index of that portion of the binder soil finer than a No. 40 sieve

By using a graphical procedure involving logrithmic coordinates, the best gradation may be determined for a particular maximum size aggregate which conforms to the Talbot equation and still contains the required quantity of clay.

Recent research.--In the past few years there have been attempts to determine the characteristics of soil-aggregate mixtures under controlled laboratory conditions. A few of them should be noted in particular.

In 1940 an investigation undertaken by Deklotz was reported in which the effects of the quantity and quality of soil binder were studied.(20) It was concluded that there is a definite relationship between the plasticity index and liquid limit and the stability of a soil-aggregate mixture. In addition, it was suggested that rejecting a mixture on the basis of plastic index and liquid limit alone, without consideration of the quantity of fine-grained material in the mixture, was in many cases uneconomical.

The Bureau of Public Roads published a report (21) in 1946 of an investigation of soil-aggregate stabilization in which the plasticity index of the binder soil was varied from nonplastic to well above the specification limit of six established by the American Society for Testing Materials.(22) The report stated that while slightly plastic soil produced higher stability than nonplastic, a plasticity index of six was the maximum that should be allowed.

The effect of varying the soil content on the compaction and strength characteristics of certain soil-aggregate mixtures was investigated by Yoder and Woods at Purdue University and reported in 1946.(23) On the basis of the test data developed, it was indicated that maximum densities do not necessarily mean maximum strengths whenever soil-aggregate mixtures with near optimum binder soil contents are considered. In these tests the California bearing ratio was used as a measure of strength. For mixtures of soil and gravel, maximum bearing values were obtained when the proportion of material finer than the No. 200 sieve reached above seven per cent. The optimum for maximum density was at ten per cent.

CHAPTER III

MATERIALS AND EQUIPMENT

The equipment and materials used in this investigation are listed and described below.

Materials

Aggregate.--The aggregate used was a coarse, slightly micaceous, natural sand with subangular to angular grains. This material was obtained from a pit located several miles northeast of Cartersville, Georgia. Its physical properties are as follows:

Specific gravity of soil solids	- 2.71
Effective size	- 0.33 millimeters
Uniformity coefficient	- 2.8
Gradation	- See Fig. 2
Plasticity	- None

Soil binder.--The soil binder used was a reddish, inorganic, sandy, silty clay of low plasticity. The material was obtained from a pit on the Georgia Institute of Technology campus. Its physical properties are as follows:

Specific gravity of soil solids	- 2.68
Effective size	- 0.0006 millimeters
Uniformity coefficient	- 133.0
Gradation	- See Fig. 2
Liquid limit	- 31
Plasticity index	6

Compaction Equipment

A U. S. standard five and one-half pound compaction hammer was employed; the ram diameter was two inches, and a free fall of twelve

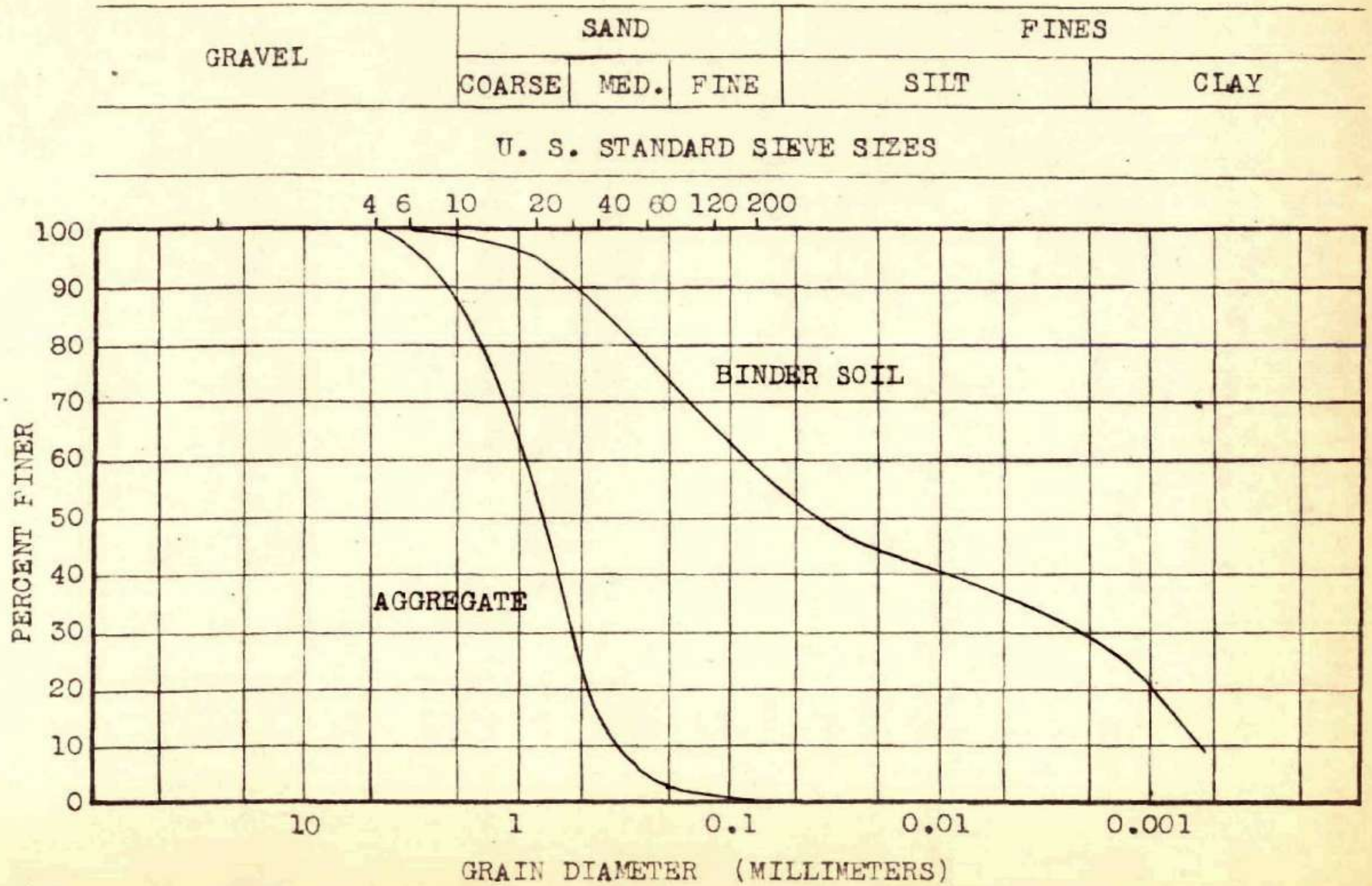


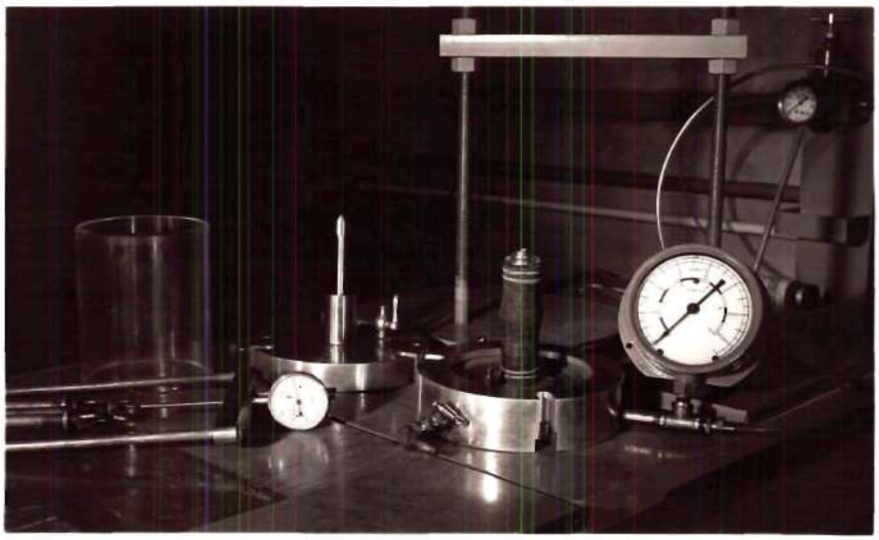
Fig. 2. Grain-Size distribution for Aggregate and Binder Soil

inches was used. The mold was a U. S. standard (Proctor) compaction mold with a nominal volume of one-thirtieth of a cubic foot. Other materials included various scales, dishes, ovens and supplies found in any well-equipped soil mechanics laboratory.(Fig. 3)

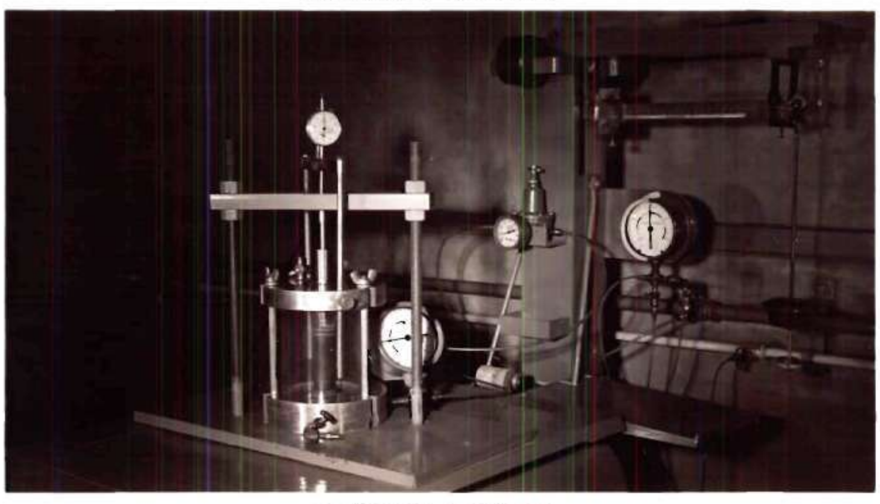
Shear Apparatus

Shear equipment consisted of a "portable" triaxial chamber and a beam type scale loading machine.(Fig. 3) The triaxial chamber could accommodate soil samples 1.4 inches or 2.8 inches in diameter at a maximum confining pressure of 100 pounds per square inch. Controlled stress incremental loading was dictated by the design of the loading machine. Maximum applied load possible was 2,000 pounds.

COMPACTION EQUIPMENT



TRIAXIAL CHAMBER



LOADING MACHINE

Fig. 3. Laboratory Equipment

CHAPTER IV

EXPERIMENTAL PROCEDURE

Preliminary.--The materials were obtained and transported to the laboratory. The soil binder was placed in large flat pans and allowed to air dry for nearly two weeks. Large lumps were broken up by hand, the material was screened through a No. 4 sieve, and the material retained was discarded. The fraction passing was thoroughly mixed in a dry concrete mixer, "quartered" and remixed to insure uniformity throughout the succeeding test program. The aggregate was mixed in the same manner. After this preparation, the materials were placed in separate large galvanized containers equipped with tight fitting lids.

Classification tests were performed on each of the two materials according to standard procedures.(24) These tests included specific gravity, grain-size distribution, and Atterberg limits. The latter was performed on soil binder only.

Preparation of the soil-aggregate mixtures.--In preparing a mixture of the two materials, the following procedure was used. Lumps in the soil binder were broken up with a mortar and rubber tipped pestle. The soil and aggregate were weighed out in quantities to produce a fifteen pound batch of the desired proportions. The aggregate was spread out in a relatively thin layer in a large pan. The soil binder was then spread out on top and thoroughly mixed into the aggregate.

A nine pound batch of each mixture was weighed out, placed in a pail fitted with an airtight lid, and set aside for the triaxial shear tests. The remaining six pounds of each batch was used for determining the optimum moisture content and the maximum dry density of the mixture.

The proportioning of the mixtures was done on the basis of moist weights of each material as it came from the stock containers. It was therefore necessary to determine the moisture content of each material each time a mixture was prepared so that the actual proportions could be calculated. Mixtures were prepared containing the percentages of soil binder as shown in Table 1.

TABLE 1.

Percentage of Soil Binder

Desired	Actual
0.0	0.0
4.0	3.6
8.0	7.3
12.0	11.1
16.0	14.8
20.0	18.4
24.0	22.8
28.0	26.2
36.0	33.8
43.0	40.8
50.0	47.6
100.0	100.0

All graphs and computations in this presentation are based on the actual mix proportions as shown in the foregoing table.

Determination of optimum moisture content and maximum dry density.--A single compaction mold was chosen for use throughout the test program. The compaction mold was calibrated volumetrically by weighing empty and again filled with water at room temperature. The volume of the mold (calculated from the weight of the water contained) was thereafter used in all of the computations.

Samples were compacted using twenty-five blows of the five and one-half pound hammer, falling twelve inches on each of three equal layers of soil in the compaction mold. Except as noted below, standard procedures were used as outlined by the American Society for Testing Materials.(25) As specified in the standard procedure, the soil was reused for each successively higher moisture content.

Water was added to the mixture in quantities approximating a two per cent increase in moisture content. As it was added, the material was turned and mixed. The moist mixture was then covered and set aside to "age", allowing the moisture to become distributed throughout. In the mixtures containing small amounts of soil binder, an "aging" period of approximately fifteen minutes was judged sufficient. The mixtures containing greater percentages of soil binder were allowed approximately an hour for the moisture to become adequately distributed.

A minimum of five samples were compacted, each at a successively higher moisture content. In most cases, however, moisture contents were increased and samples compacted until the mixture became unworkable. Usually this resulted in six or seven determinations.

From the information thus obtained, a plot of moisture content versus maximum dry density was developed. (Hereafter this relationship is referred to as a moisture-density curve.) At the maximum dry density, the corresponding moisture content (optimum) was determined. Moisture-density curves for each mixture appear in the Appendix.

Preparation of triaxial test specimens.--Sufficient water was added to the remaining nine pound batch of each mixture to bring the material to the previously determined optimum moisture content. After the water had been added and thoroughly mixed in, the material was placed in an airtight container and allowed to "age" as in the moisture-density determinations. After the required elapsed time, a representative sample was taken to determine the moisture content. An adjustment was made as dictated by this determination (either by air drying the mixture or by adding small quantities of water) until it fell within acceptable limits. A tolerance of one-half of one per cent was attempted; however, variations in moisture content down to one and one-half per cent on the low side were considered acceptable. Accurate control of the moisture content was quite difficult in the mixtures containing the lower percentages of binder soil. The larger variations in moisture content occurred in these mixtures.

After the moisture content had been adjusted, a sample was compacted in the compaction mold using the same procedure as described in the foregoing subparagraph. The soil and mold were weighed to determine density. The compacted sample was then extruded from the mold using a hydraulic loading machine, and a triaxial shear specimen was immediately prepared from the sample.

A 1.4 inch diameter tubular specimen cutter, approximately three inches long, was placed on top of the compacted material. The hydraulic loading machine was used to maintain a small but constant downward pressure on the specimen cutter as the excess material was trimmed away from the outside. The trimmings were recombined with the remaining stock of the particular mixture.

Triaxial shear tests.--Both ends of the prepared triaxial shear specimen were covered with porous stone discs. The specimen was extruded from the tubular cutter, placed on the triaxial chamber base, and encased in a thin, airtight, rubber membrane. The chamber was assembled, placed in the loading machine, and a micrometer dial positioned to indicate the axial deformation. Compressed air was admitted to the chamber to provide a confining pressure on the test specimen. Usually three triaxial shear tests were run on each mixture. These were at confining pressures of five, fifteen, and thirty pounds per square inch.

The axial stress was increased in increments to provide a minimum of ten points prior to reaching the failure stress. The loads were increased at intervals of thirty seconds, and the axial deformation recorded at fifteen seconds after each stress increase. Failure of the specimen was said to have occurred when either a sudden shear took place or it became impossible to maintain the load on the sample throughout the thirty second loading interval.

From the deformation data recorded during the triaxial test, the average cross-section area of the sample and the applied stress were computed for each load increment, assuming no soil volume change during loading. The strain of the sample under each load was calculated and a stress-strain curve drawn. From this curve the ultimate stress imposed by the loading machine was determined. In the majority of cases the stress-strain curve had to be extrapolated a short distance to determine this point.

From the test values, Mohr's circles were drawn; and from all the Mohr's circles for a given mixture, a Mohr's envelope was constructed. In this report, the angle between the Mohr's envelope and the horizontal is defined as the angle of internal friction, ϕ. The intercept of the Mohr's envelope at the vertical axis is the value of apparent cohesion, C. Occasionally there were membrane leaks, equipment malfunction, or other specific difficulties. In these instances individual tests were disregarded or repeated.

Vacuum shear tests.--Since it is impossible to trim a sample from cohesionless material, compression tests on the mixture containing zero per cent soil binder were performed as vacuum shear tests. In this test series, a representative sample of the aggregate was dried in an oven at 100°C. A thin rubber membrane was fastened to a base, stretched over a tubular forming jacket and filled with the aggregate. The surface was leveled, a bearing cap placed on top, and the membrane turned up and fastened. A partial vacuum was imposed on the interior of the sample providing a confinement equal to the differential air pressure between the interior of the sample and the atmosphere. The sample was stressed in the same manner as in the triaxial shear tests. The center diameter of the sample was measured by micrometer calipers after each load increase. A stress-strain curve was plotted, ultimate strength determined, and Mohr's circle drawn. Since in a cohesionless material the Mohr's envelope passes through to origin of the coordinates, an envelope could be constructed from only one test.

In preparing the sample, the weight of the material used was carefully recorded, and just prior to the test, measurements were made of the diameter and length to determine the exact volume of the sample. From this information the void ratio was calculated.

A total of four vacuum shear tests were run, each with the sample prepared in a slightly different manner. In the first test, the aggregate was merely poured into the forming jacket in loose, even layers. In another test, the material was tamped lightly with a rubber tipped tamping rod. In the third test, the aggregate was tamped very firmly with the tamping rod; and for the fourth test, the wand of a portable concrete vibrator was brought into momentary contact with the exterior of the forming jacket to produce as dense a sample as practicable.

From the results of these four tests, a plot of ϕ versus void ratio was developed. This plot was extrapolated to yield a value of ϕ associated with the void ratio of the aggregate at its maximum dry density as determined by the moisture-density curve.

CHAPTER V

DISCUSSION OF RESULTS

Test results.--A tabular summary of the test results and a portion of the computed data appears in the Appendix as Table 2. The test data is discussed under the general categories of density, gradation, strength, and volume relationships. The meaning of the results of the entire test program is then discussed and is followed by an interpretation of the results in terms of bearing capacities under vehicular wheel loads.

Moisture-density relationships.--The analysis of the moisture-density relationships began with the plotting of a curve of dry density versus moisture content for each of the soil-aggregate mixtures and for the aggregate and binder soil separately. These curves appear in the Appendix as Fig. 20 through Fig. 25. Hereafter in this presentation, the density of a particular soil-aggregate mixture refers to the maximum dry density determined from the moisture-density curve for that particular mixture. The optimum moisture content is the moisture content at which the maximum dry density occurs.

For a comparison of the series of mixtures, the densities and optimum moisture content of each were plotted against the proportion of binder soil in the mixture. These relationships appear as Fig. 4. Inspection of these curves reveal that the mixture containing 26 per cent binder soil was compacted to the highest density. This mixture also had the lowest optimum moisture content.

The curve of density versus percentage of binder soil in the total mixture is quite steep, between zero per cent and 4 per cent binder soil,

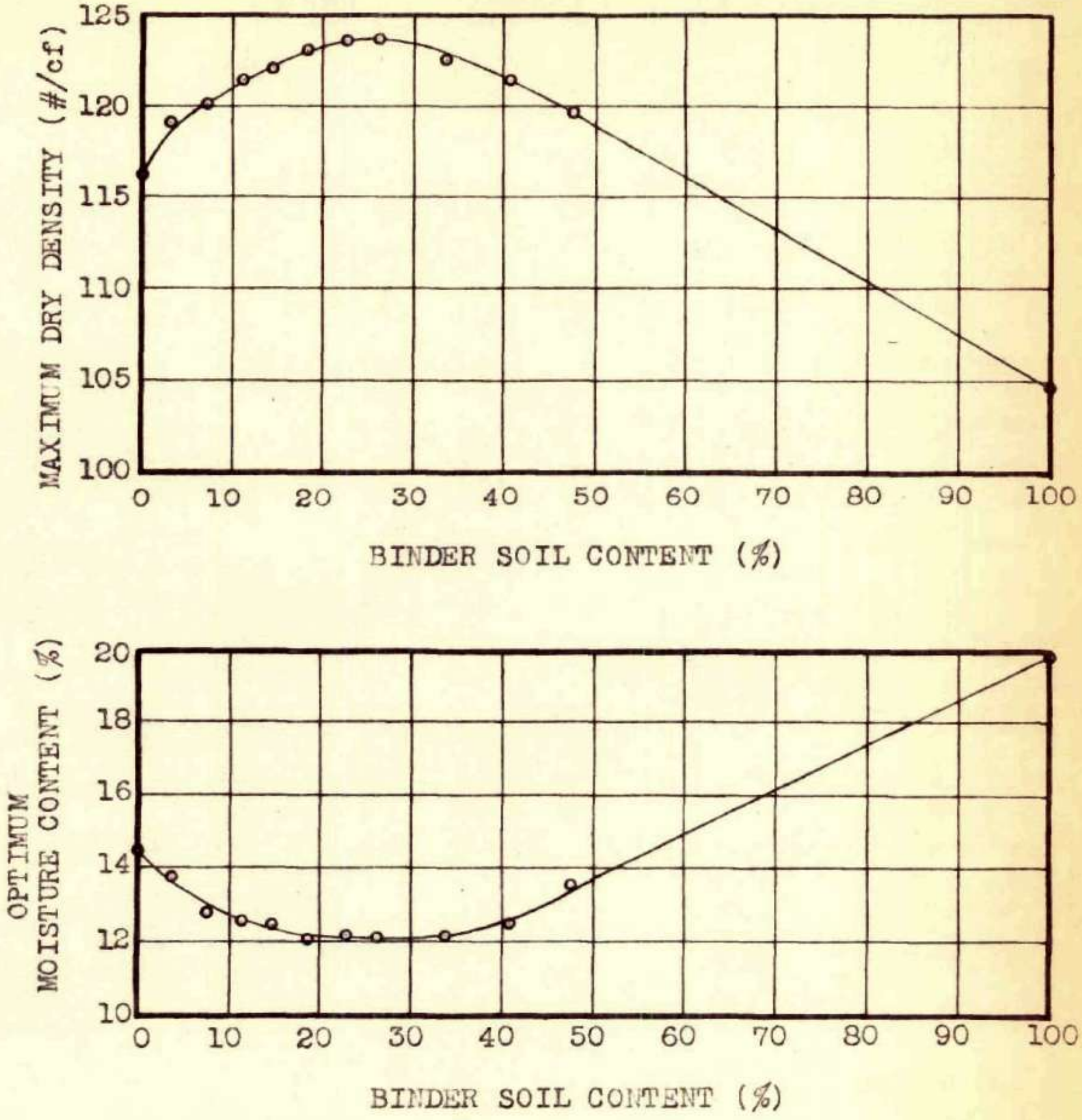


Fig. 4. Relation of Maximum Dry Density and Optimum Moisture Content to Binder Soil Content

becoming less steep but then remaining at very nearly the same slope for binder soil proportions of from 4 per cent to 23 per cent. The initial steep slope may be attributed to a slight lubricating action of the first small quantities of binder soil. This action would allow the grains of aggregate to move more easily into a more dense condition under the same compactive effort. When binder soil is present in an amount in excess of that required for this lubrication action, the density increases at a nearly constant rate, until it approaches the mixture with 26 per cent binder soil. At mixture proportions greater than 26 per cent, the densities decrease at a nearly uniform rate to that of the binder soil alone. The optimum moisture contents also vary, decreasing from 14 to 12 per cent, then increasing to 20 per cent (that of the binder alone).

Gradation.--Gradations for the aggregate, the binder soil, and each of the ten intermediate mixtures are presented in Fig. 5. Not all of the gradation curves have been plotted in their entirety due to space limitations. It can be seen that the largest variation occurs in the medium and fine sand sizes. Lesser changes occur in the proportions of silt and clay sized particles respectively, and even less in the coarse sand.

Superimposed on these gradations are the gradation limits established by the American Society for Testing Materials for type E soil-aggregate mixtures. It can readily be seen that mixtures containing from 11 per cent through 34 per cent binder soil will meet the specifications for gradation.

The gradation for the mixture with the greatest density (26 per cent binder soil) appears in Fig. 6. Compared with it are gradations computed by the Talbot equation when "n" equals 1.0 (a straight line plot on arithmetic coordinates) and where "n" equals 0.5 (a parabolic curve on

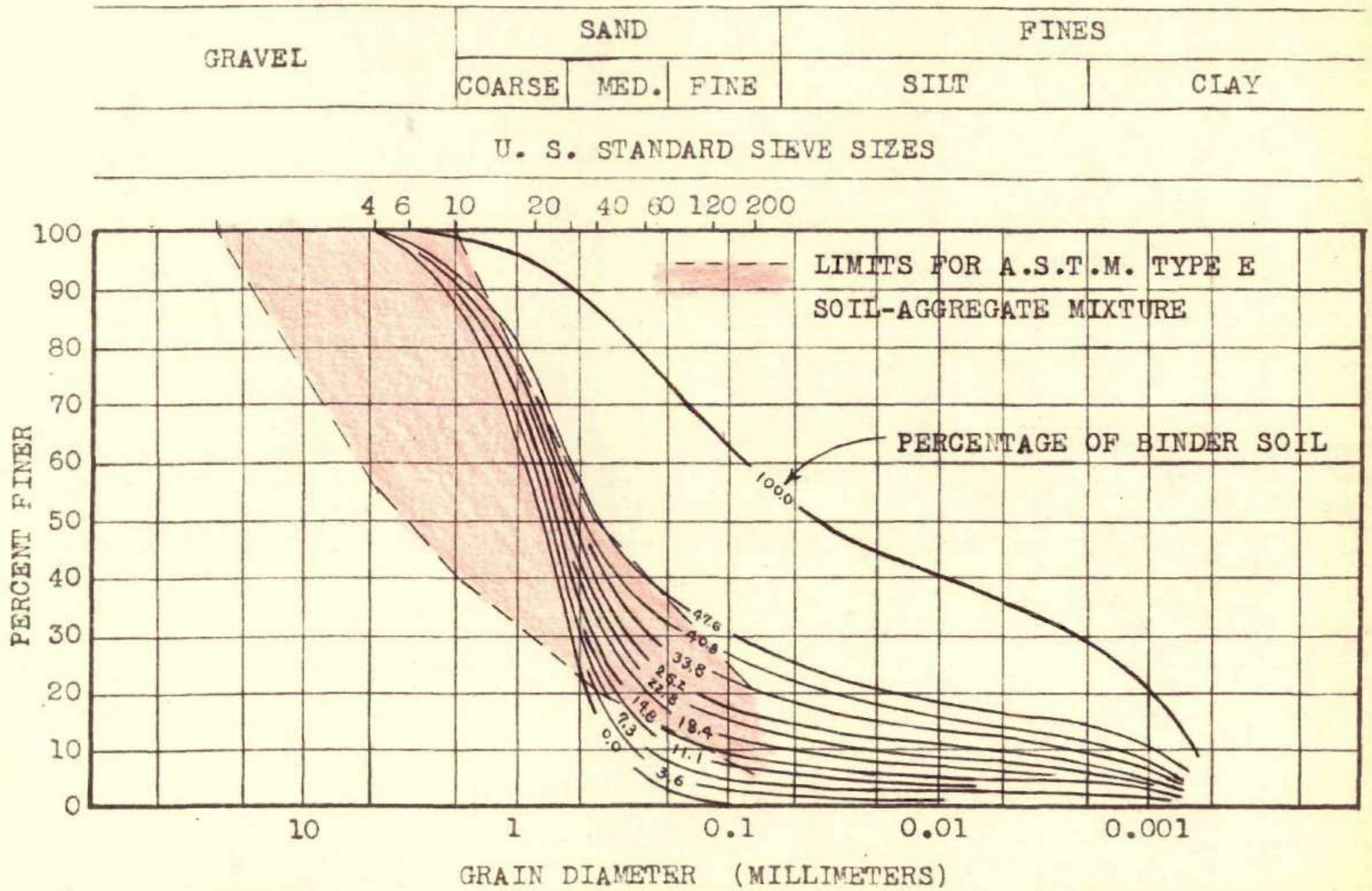


Fig. 5. Grain-Size Distribution for Variations in Binder Soil Content

arithmetic coordinates). In addition, the "best" gradation was determined by the method used by Hennes for the same maximum size aggregate and the same percentage of clay sized particles as in the mixture with the greatest density.(19) This curve can be described by the Talbot equation with a value of "n" equal to 0.33.

These same curves are replotted on logarithmic coordinates and presented as Fig. 7. In this manner, all variations of the Talbot equation become straight lines. A comparison of the theoretical and actual appears to be less difficult in this case. From Fig. 7 it appears that the curve for the mixture with the maximum density most nearly approximates the theoretical gradation with the value "n" equal to 0.33 (the Hennes method). The "perfect" gradation described by the often-used parabolic form of the Talbot equation is most closely approximated by the mixture containing only 11 per cent binder soil. This mixture had a compacted density about two pounds per cubic foot less than the mixture containing 26 per cent binder soil.

Further study of the gradation curve for the mixture with 26 per cent binder soil suggests that one of the greatest deviations from the "perfect" gradation is coarser material. Comparison with "perfect" gradations for maximum size aggregates corresponding to the No. 4, No. 8 and No. 14 sieves (Fig. 8) indicates that perhaps the gradation based on the No. 8 sieve more closely approximates the actual mixture. An inspection of the gradation represented on logarithmic coordinates (Fig. 9) reinforces this choice. None of the theoretical curves come close to the actual gradation, however.

<u>Strength characteristics</u>.--The analysis of the strength characteristics of the various mixtures commenced with plotting stress-strain curves and

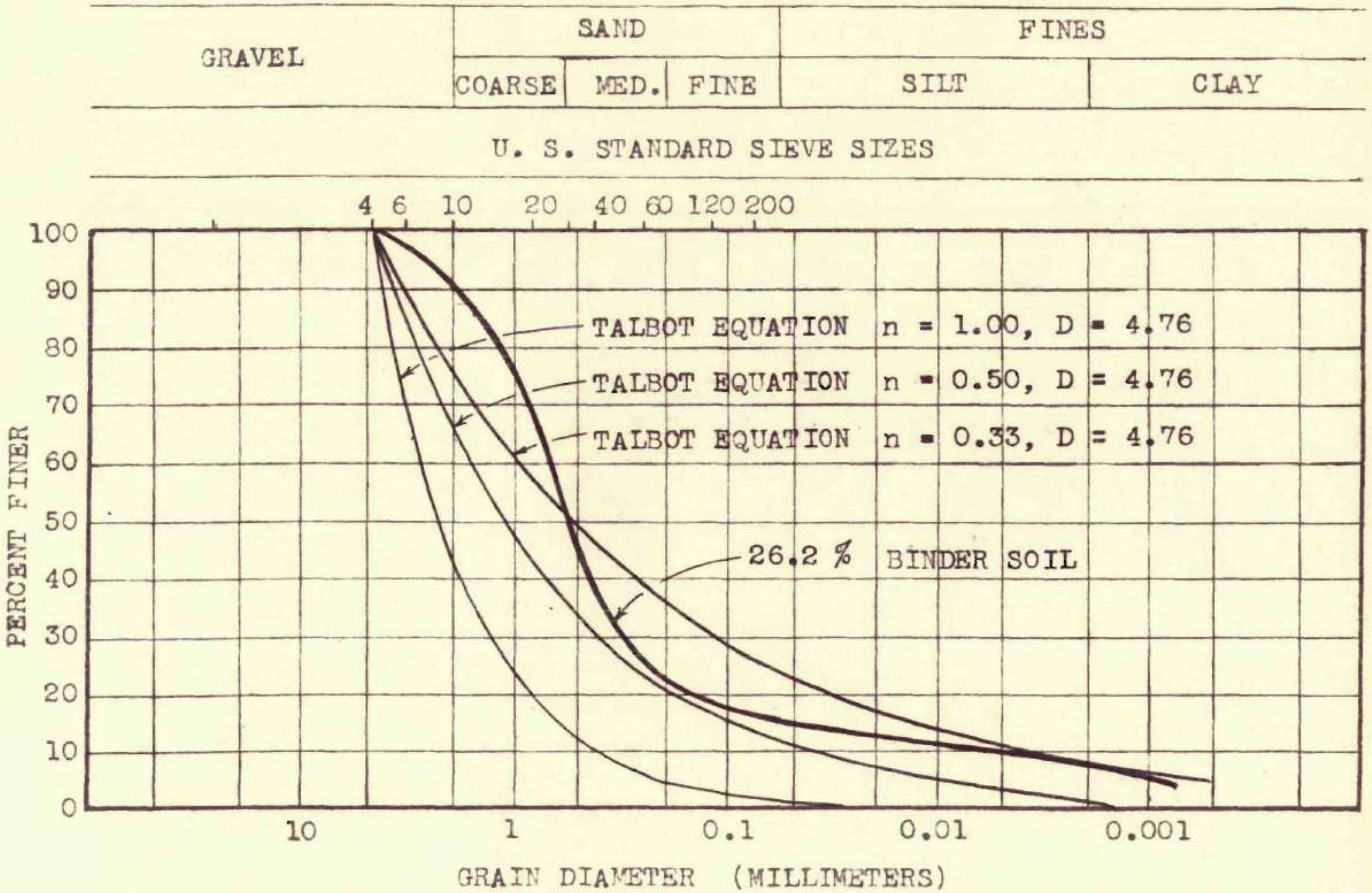


Fig. 6. Gradation of Mixture Containing 26 Per Cent Binder Soil and "Perfect" Gradations With Variations in the Value of "n".

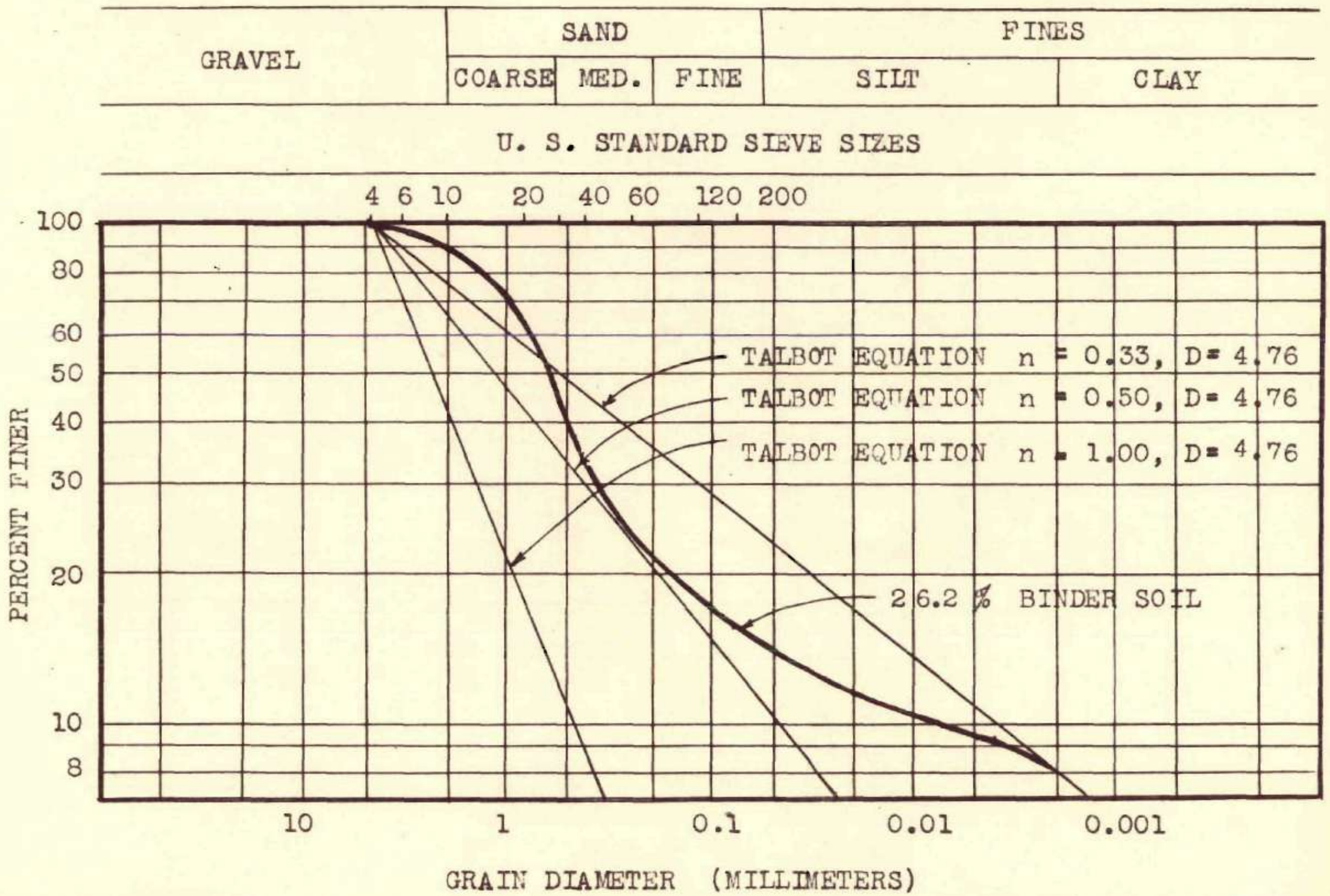


Fig. 7. Gradation of Mixture Containing 26 Per Cent Binder Soil and "Perfect" Gradations With Variations in the Value of "n".

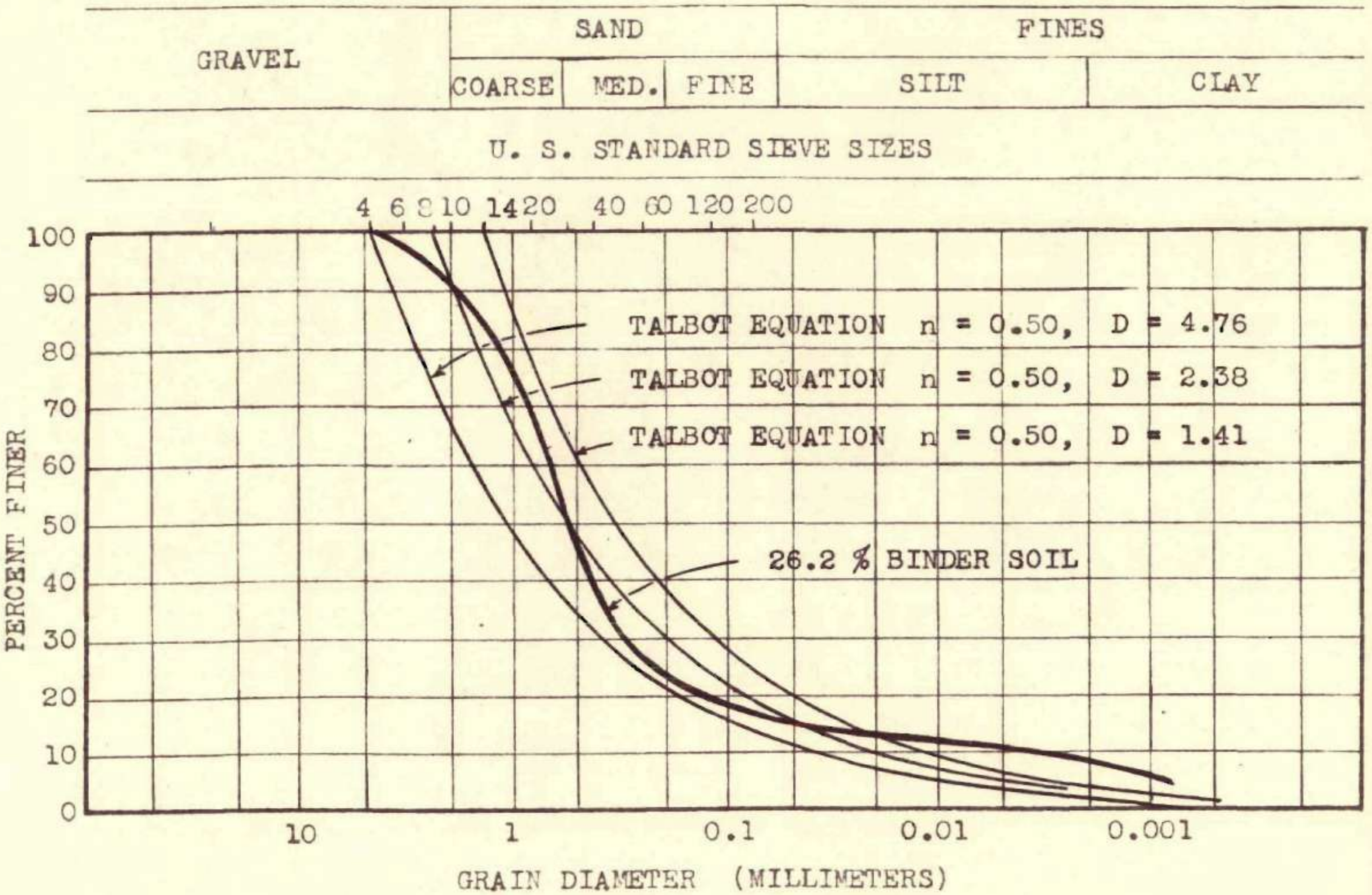


Fig. 8. Gradation of the Mixture Containing 26 Per Cent Binder Soil and "Perfect" Gradations With Variations in the Value of "D".

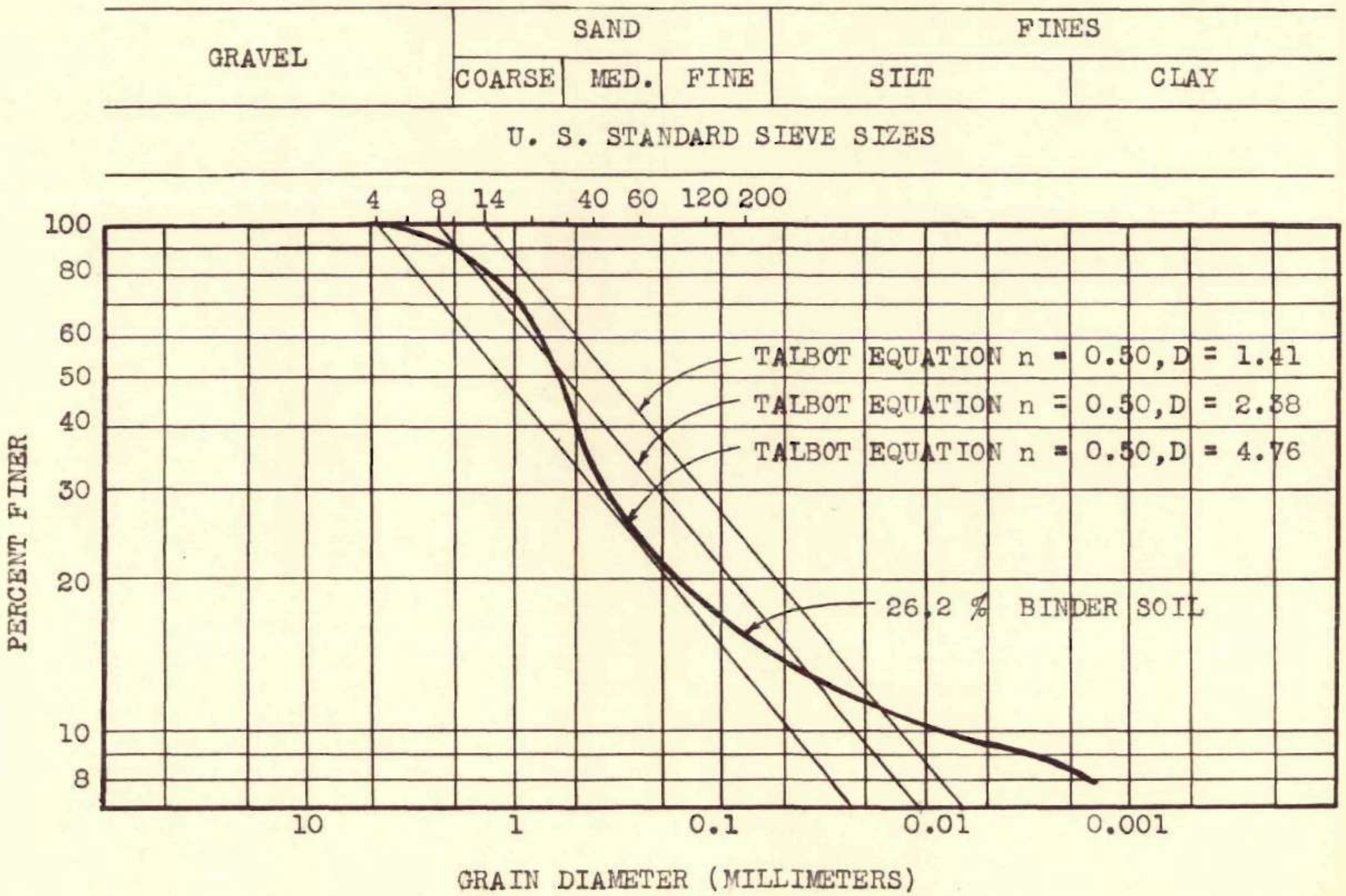


Fig. 9. Gradation of the Mixture Containing 26 Per Cent Binder Soil and "Perfect" Gradations With Variations in the Value of "D".

Mohr's circles for the triaxial tests. Mohr's envelopes were constructed and the value of cohesion (C) and the angle of internal friction (ϕ) determined. Stress-strain curves, Mohr's circles, and the Mohr's envelope for each mixture are presented in the Appendix as Fig. 26 through Fig. 37. Figure 26, which presents data for the cohesionless aggregate, also includes a plot of the angle of internal friction versus void ratio, which is necessary for the analysis of vacuum shear tests on cohesionless soils.

The values of cohesion and the angle of internal friction are shown by Fig. 10 as a function of the proportion of binder soil in the mixture. From this relationship, several occurrences may be noted. The aggregate alone exhibits the high angle of internal friction (45.7 deg.) characteristic of dense, angular, cohesionless material. The binder soil alone demonstrates considerable cohesion and a much lower angle of internal friction (23.0 deg.) as is typical for sandy, silty clays that are partially saturated.

The addition to the aggregate of a very small quantity of the binder soil resulted in a sharp decrease in the angle of internal friction and the appearance of cohesion. As the proportion of binder soil was increased from 4 per cent to 26 per cent, gradual changes occurred in both the cohesion and angle of internal friction. The cohesion increased to a value of approximately 0.75 kips per square foot, while the angle of internal friction decreased to a value of 35 deg.

Increasing the proportion of binder soil in the mixture from 26 per cent to 34 per cent brought about a very marked change in the strength characteristics. The angle of internal friction dropped to about 23 deg., but the cohesion more than doubled, increasing to a value of 1.7 kips per square foot. From this point, further increases in binder soil did not

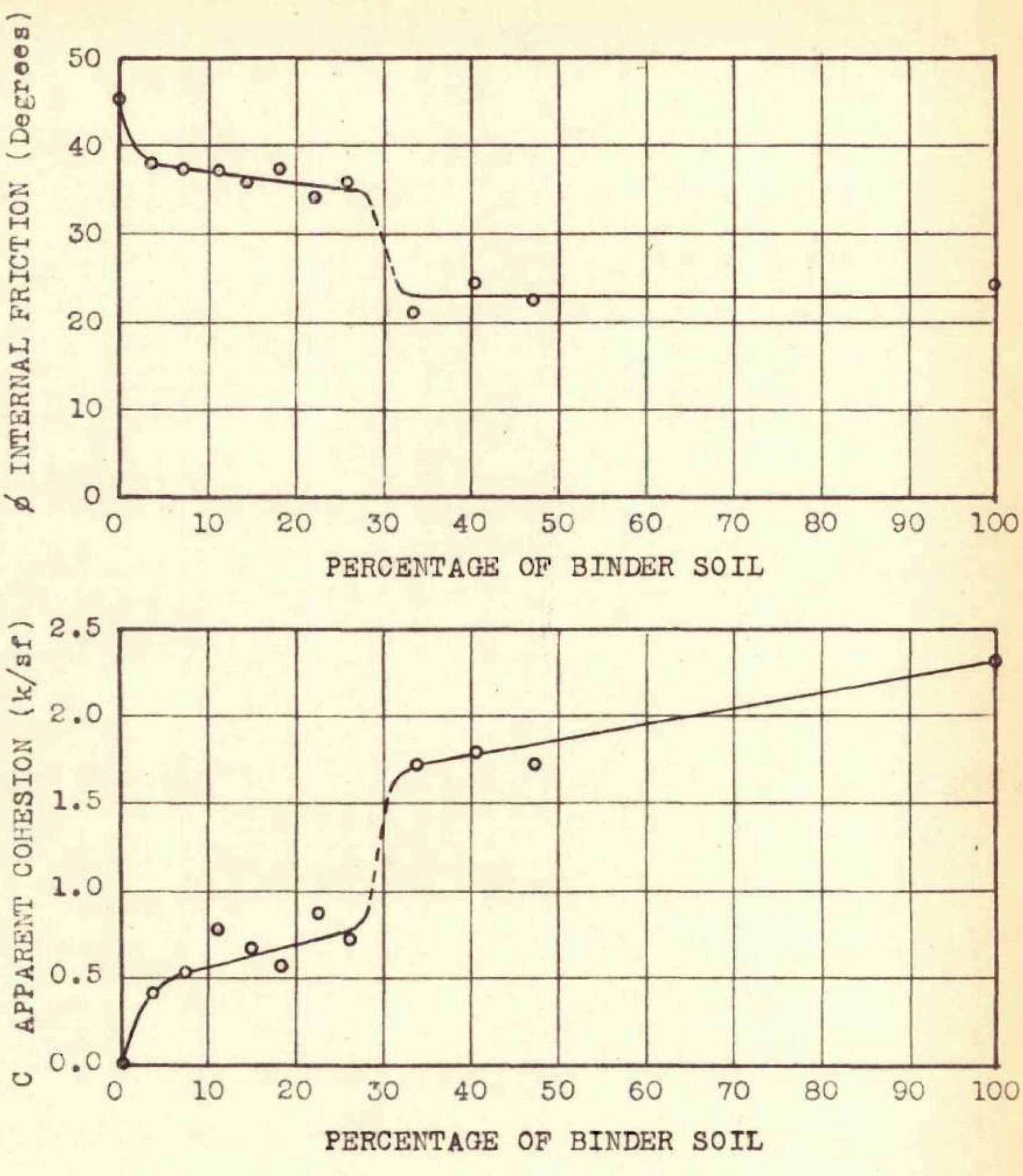


Fig. 10. Internal Friction and Apparent Cohesion Versus the Percentage of Binder Soil

affect the angle of internal friction. The value remained at 23 deg., equal to that of the binder soil alone. Cohesion, however, continued to increase at very nearly the same rate as before, ultimately reaching the value of cohesion for the binder soil alone, which is 2.3 kips per square foot.

Since it is common practice to consider soil particles smaller than 0.002 millimeters in diameter as being cohesive, and those particles larger than that as cohesionless, the values of cohesion and the angle of internal friction have been plotted as a function of the percentage of particles smaller than this size. This relationship appears as Fig. 11. The same marked change in the character of the mixtures is displayed, with the change occurring at a "clay" content of about 8 per cent.

Volume relationship.--The series of block diagrams shown as Fig. 12 portray variations in the volume of aggregate, binder soil, water and air between the various mixtures. By comparing the diagrams for the aggregate alone and the mixture containing 4 per cent binder soil, it can be seen that the increase in the total volume of soil solids in the latter mixture is not equal to the volume of binder soil. The addition of binder soil apparently displaced some of the aggregate particles, even though the binder soil represented only 8 per cent of the volume of voids in the aggregate. This indicates that even when a very small quantity of cohesive material is present in a mixture, it has a tendency to coat grains and hold them slightly apart. The sharp drop in the angle of internal friction and the appearance of substantial cohesion at 4 per cent binder soil content is another indication of this action.

As can be seen from the block diagrams, the volume of entrapped air remains essentially constant throughout the series of mixtures. Variations

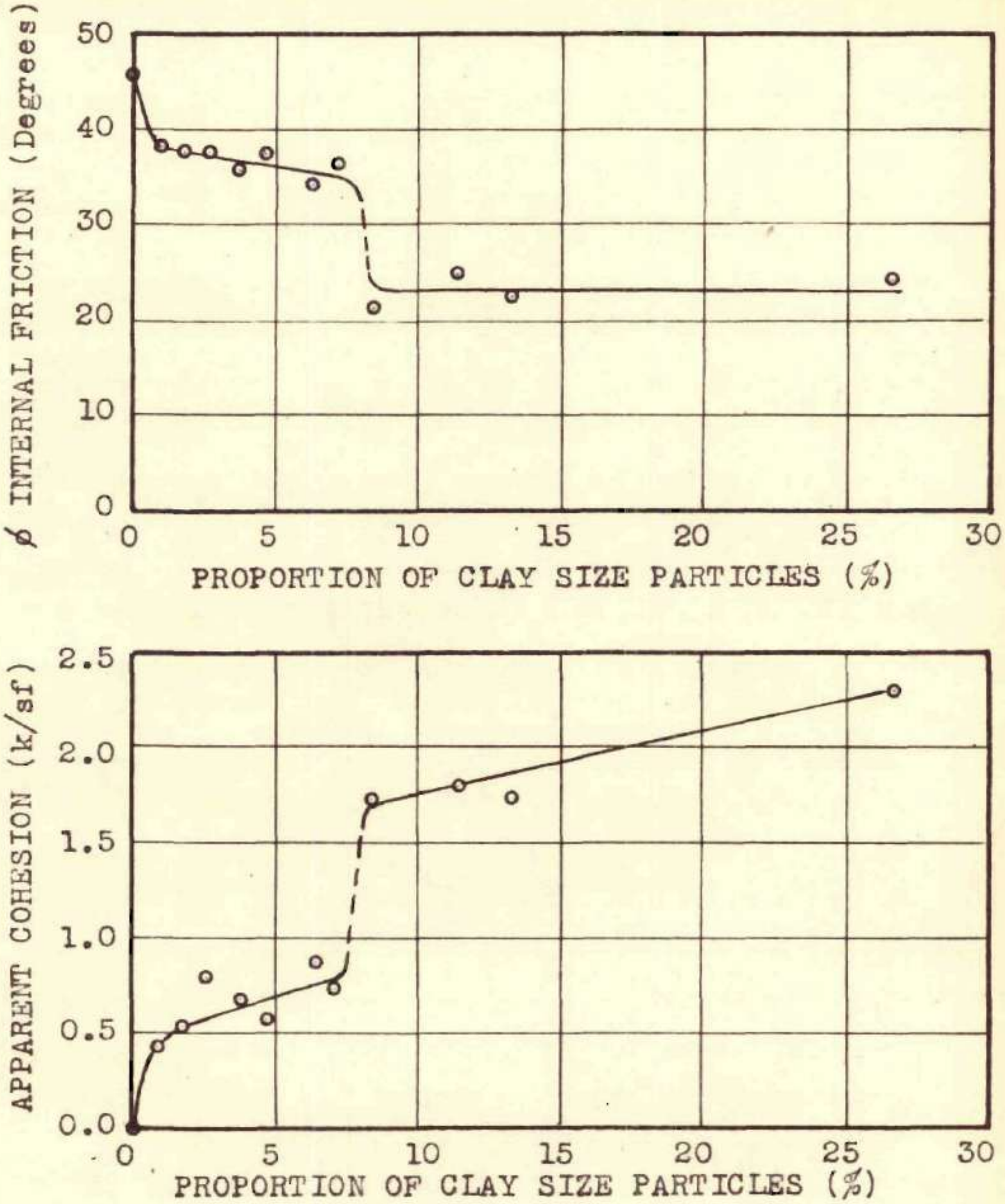


Fig. 11. Internal Friction and Apparent Cohesion Versus the Proportion of Clay Size Particles

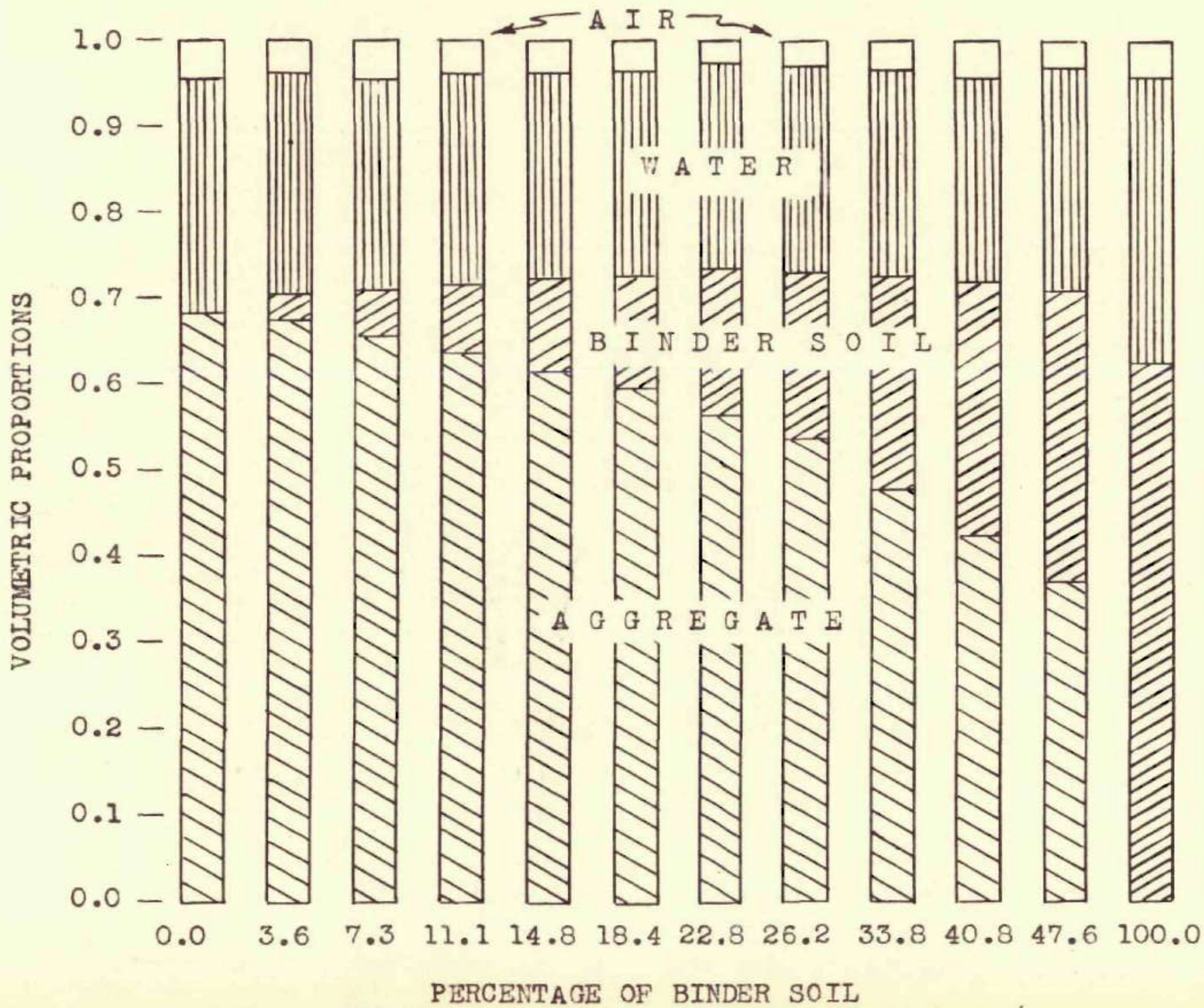


Fig. 12. Aggregate, Binder Soil, Water, and Air Volume Relationships

in the degree of saturation, (which is closely related to the volume of entrapped air) are presented in Fig. 13. There is a slight trend toward higher saturations in the more clayey mixtures. The values of saturation for the various mixtures show a variation of several per cent, and therefore it is difficult to draw conclusions of anything other than a slight trend.

A comparison between the volume of the voids in the sand alone and the volume of voids in the total mixture is afforded by the two curves shown in Fig. 14. The voids in the total mixture reach a minimum in the mixture containing 26 per cent binder soil. This is consistent with the density relationships shown in Fig. 4. The significant thing about Fig. 14 is that the voids in the sand alone increase as a curvilinear function for the range of mixtures containing zero per cent to about 26 per cent binder soil. For mixtures with a greater proportion, a straight line relation exists. This indicates that in mixtures of over 26 per cent binder soil, additional binder displaced a like amount of the aggregate, and that the binder was filling the maximum void space possible under that particular compactive effort.

Figure 15 shows the percentages of the available void space in the sand that are filled with binder soil. The voids are filled at a constant rate in the series of mixtures from zero per cent to 15 per cent binder soil. In mixtures with higher binder contents, the percentage of voids filled increases but at a decreasing rate. This occurrence is more discernable on the logarithmic coordinates.(Fig. 16) From zero per cent to 15 per cent binder soil the granular material apparently absorbs the great majority of the effort expended in compacting the mixture. From 15 per cent to about 25 per cent the slope of the line is less, indicating that the voids are being filled at a lower rate. In this transition region,

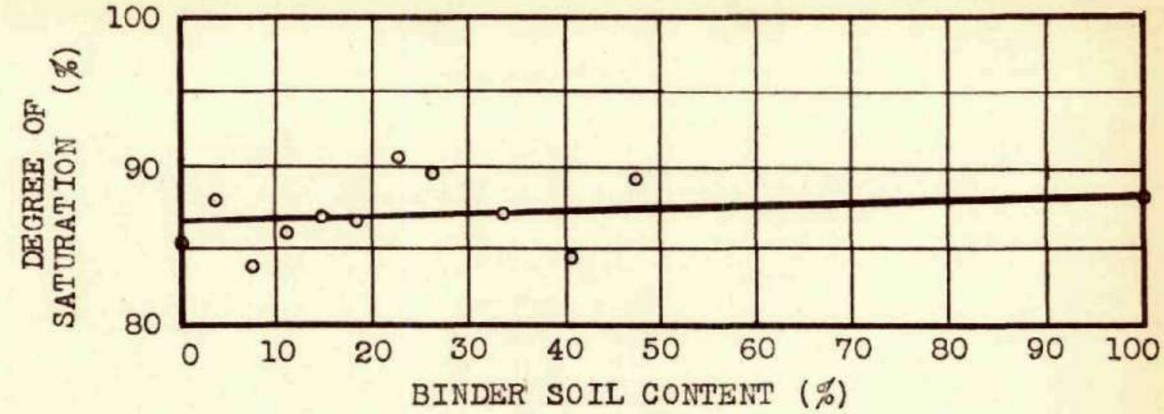


Fig. 13. Degree of Saturation Versus Binder Soil Content

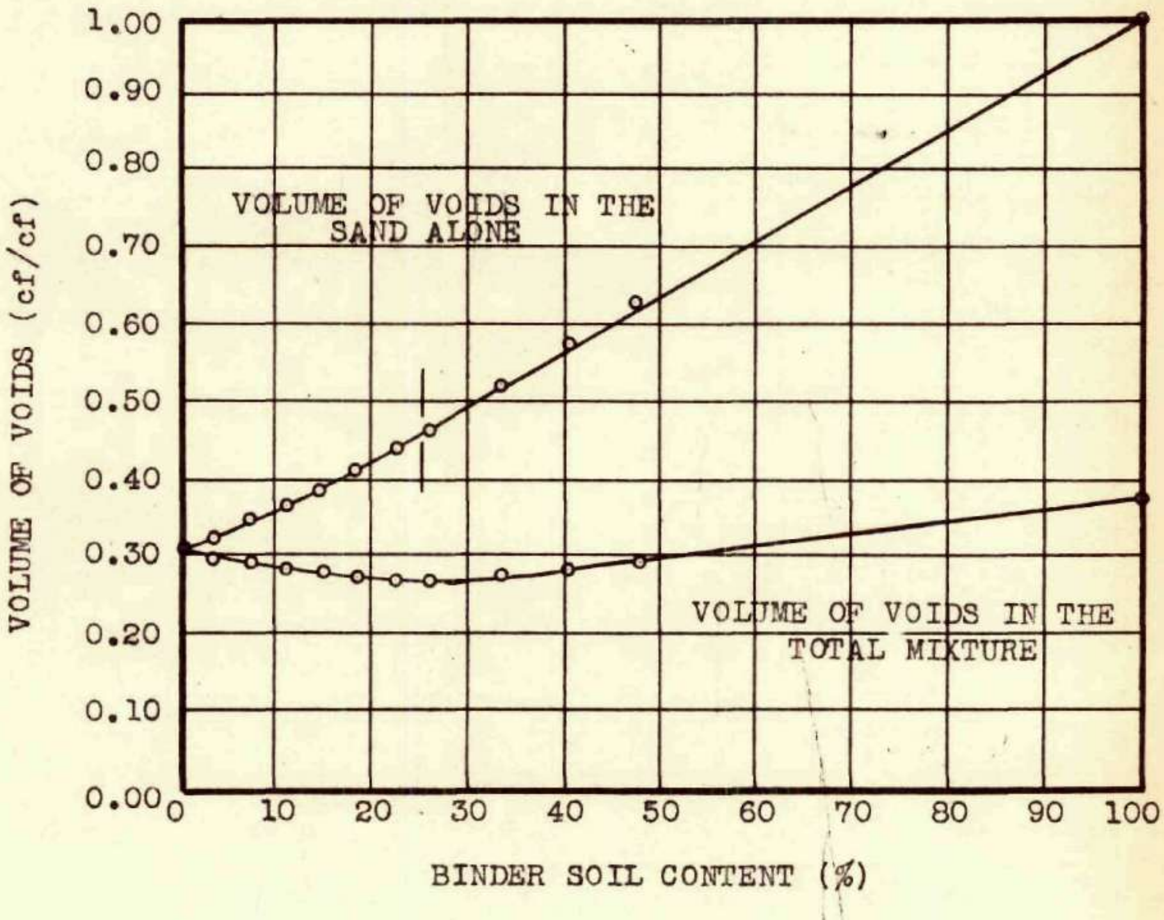


Fig. 14. Volume of Voids Versus Binder Soil Content

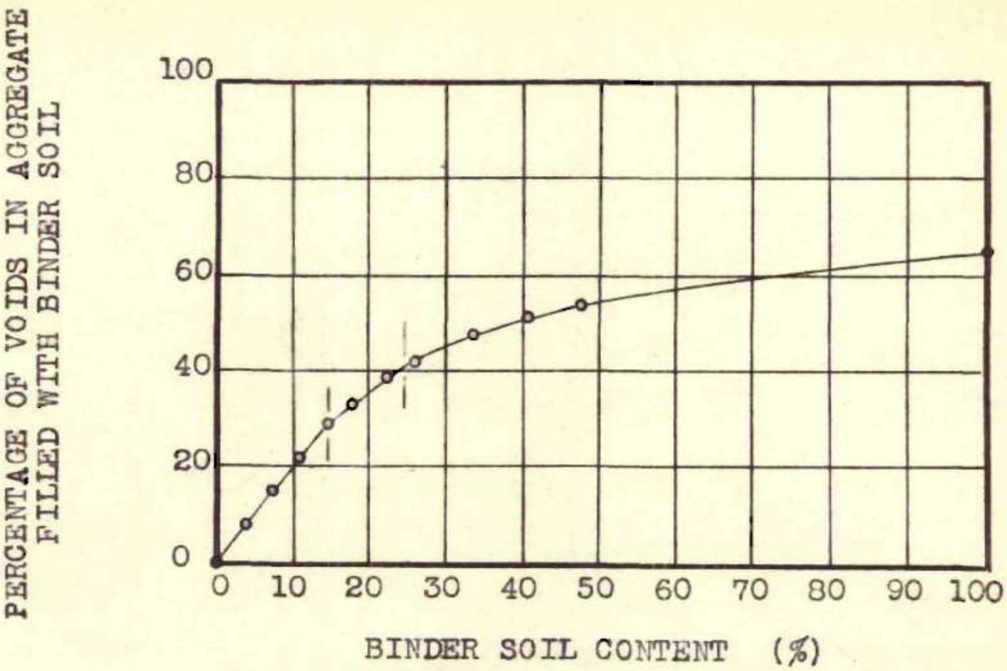


Fig. 15. Percentage of Voids in Aggregate Filled With Binder Soil Versus Binder Soil Content

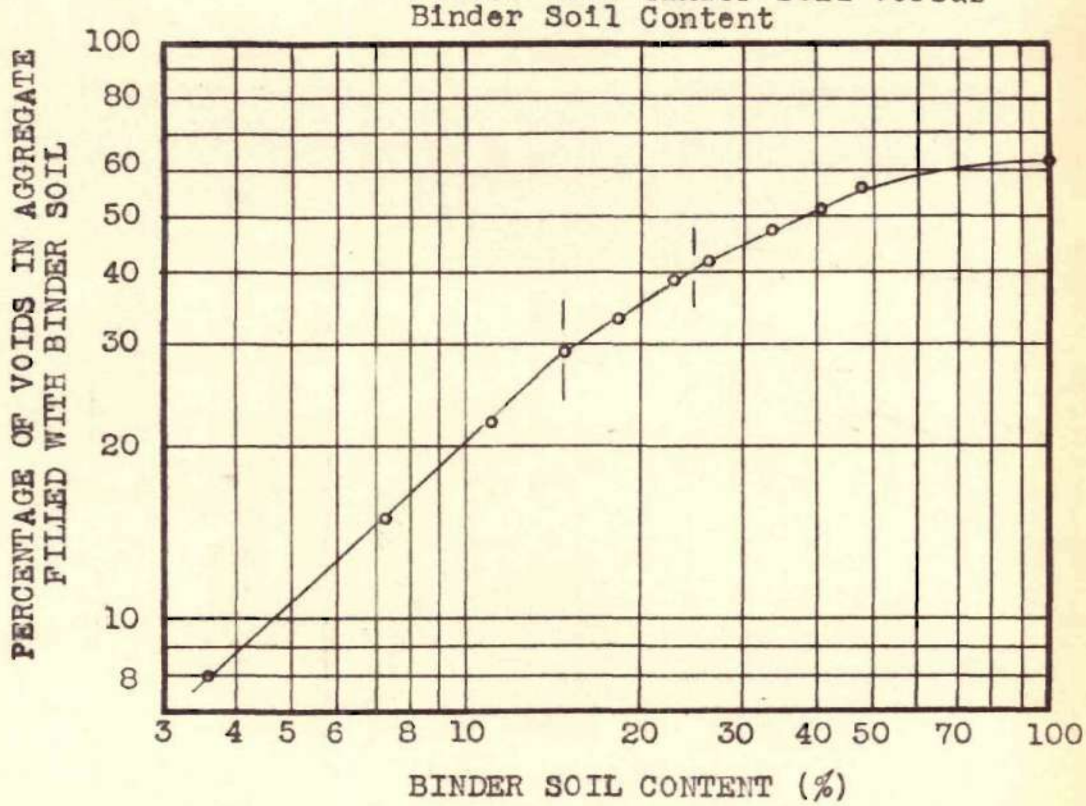


Fig. 16. Percentage of Voids in Aggregate Filled With Binder Soil Versus Binder Soil Content

loose binder "fills" the void spaces but is compacted just enough to allow intergranular contact to become established. At binder soil contents greater than 25 per cent, the rate at which the voids are filled is lower still. From 25 per cent and continuing on to the mixture containing 100 per cent binder soil, most of the compactive effort is expended in compacting the binder soil. It is significant to note that the second break in the slope corresponds to the mixture proportions at which the abrupt change in strength characteristics occur. To extend this reasoning, an increase in cohesion is expected to occur between 15 and 25 per cent binder soil content due to the partial compaction of the binder soil. This increase is not apparent however, and therefore this hypothesis may be incorrect.

In Fig. 17 the strength characteristics of the mixtures are shown as a function of the percentage of voids in the aggregate that are filled with binder soil solids. The same sharp change in both cohesion and internal friction are evident, as it was in Fig. 10. The transition zone occurs when the voids are about 45 per cent filled with binder soil solids. This emphasizes the fact that characteristics of actual soil-aggregate mixtures are slightly different from those suggested in the idealized concept presented in Chapter II.

Figure 18 presents the relationship between the percentage of water in the various mixtures which is in excess of that required for optimum moisture content in the binder soil and the binder soil content. The relation shows a definite break at a binder soil content of about 40 per cent. This break point apparently cannot be associated with other changes in the mixture characteristics, and the author presents no explanation for it.

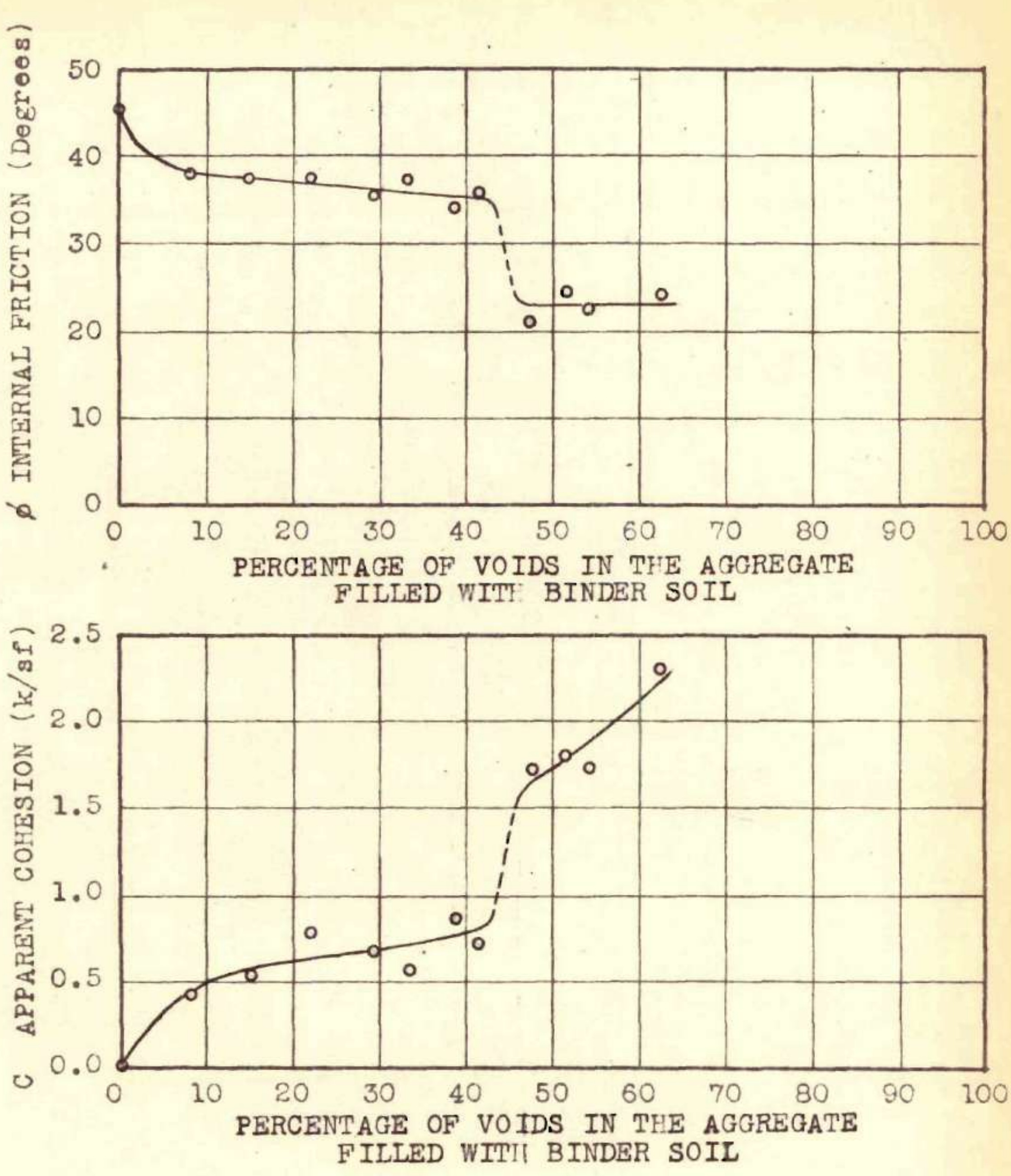
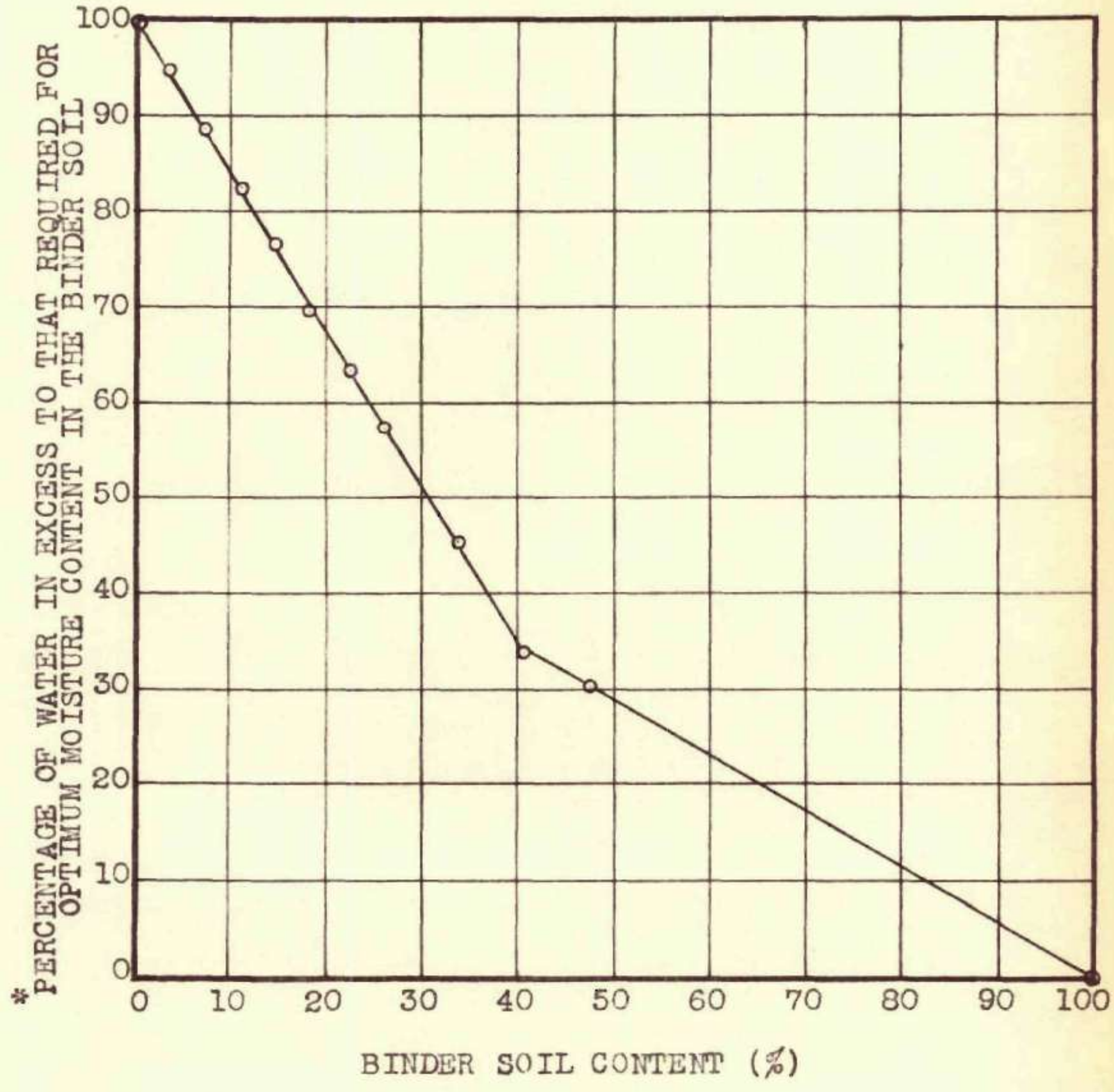


Fig. 17. Internal Friction and Cohesion Versus the Percentage of Voids in the Aggregate Filled With Binder Soil



$$^{*}\text{PERCENTAGE} = \frac{\text{EXCESS WATER}}{\text{TOTAL WATER}}\ 100\ \%$$

Fig. 18. Percentage of Water in Excess to That Required for Optimum Moisture Content in the Binder Soil Versus Binder Soil Content

Strength characteristics expressed in terms of bearing capacity.--Bearing capacities have been computed for each mixture, based on the expression shown below,(26) and using the values of internal friction and cohesion taken from the smooth curves in Fig. 10.

$$q = \gamma\, (b/2)\tan^5\alpha + 2.6\, c^1(\tan^3\alpha + \tan\alpha) + q^1\tan^4\alpha$$

The bearing capacities are shown in Fig. 19 for a loaded area equivalent to a single vehicular wheel. The curves portray situations wherein the various soil-aggregate mixtures are used as (a) surfacing, (b) base course, and (c) subgrade as defined by the explanatory sketch. It can be seen that the addition of binder soil to an aggregate used as a surface material has a very great effect, even when relatively small quantities of the binder are employed. The addition of binder has less effect in base coarse and subgrade materials, however it is of definite benefit. A proportion of binder soil in excess of 26 per cent results in a bearing capacity of much less than that for either the aggregate or binder. The bearing capacities in the 34 per cent, 41 per cent, and 48 per cent mixtures are less than that of the clay alone, probably because a large portion of the binder soil has been displaced by grains of the aggregate. This reduces the total cohesive force available in any given plane through the soil mass.

The mixture which results in the greatest bearing capacity (26 per cent binder soil) meets the gradation specifications set by the American Society for Testing Materials for Type E soil-aggregate mixtures. Mixtures with binder soil contents up to about 36 per cent will also meet these specifications. However, it is in this range of binder soil contents that an extreme loss in bearing capacity is experienced.

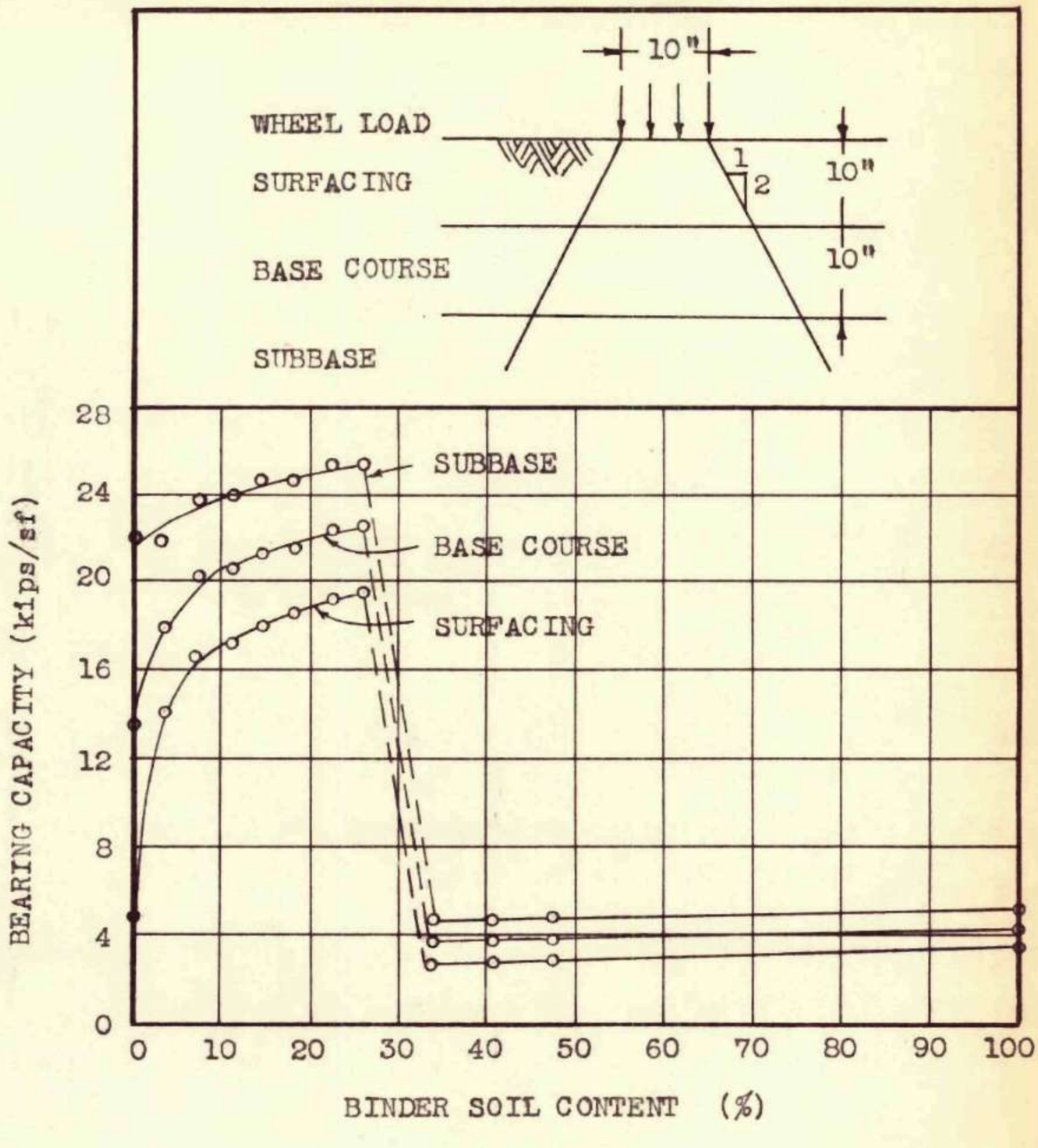


Fig. 19. Bearing Capacity Versus Binder Soil Content

CHAPTER VI

CONCLUSIONS

The following conclusions have been reached as a result of this investigation:

1. In a soil-aggregate mixture there is a certain binder soil content at which the highest maximum dry density will occur, and at which the optimum moisture content will be a minimum.
2. The gradation of the total mixture found to have the maximum density does not necessarily approximate the often-used expression for maximum density:

$$P = (d/D)^{0.5}$$

3. The addition of small quantities of binder soil to an aggregate results in definite improvement in the strength characteristics of the mixture.
4. The addition of excessive amounts of binder soil to an aggregate results in a mixture of lower strength than either of the component materials alone.
5. The addition of binder soil, even in small quantities, results in a sharp drop in the angle of internal friction, apparently due to the coating of individual aggregate particles with cohesive material.
6. A transition zone exists within which loose binder soil "fills" aggregate voids, but is compacted to permit the establishment of intergranular contact.

7. There is a possible relationship between the strength characteristics of soil mixtures and the percentage of the voids in the aggregate which are filled with binder soil.

8. Mere conformance to A.S.T.M. gradation specifications will not insure a soil-aggregate mixture with acceptable strength characteristics.

9. Optimum binder soil content for strength appears to be the same as the optimum binder soil content for maximum density.

10. The greatest benefit of stabilization by the addition of binder soil to an aggregate is realized in roadway materials to be used immediately under the loaded area.

11. The greatest increase in bearing capacity is derived by increasing the proportion of binder soil from zero per cent to 7 per cent. Smaller increases in bearing capacity result from increasing the binder soil content from 7 per cent to 26 per cent.

12. The addition of binder soil in excess of 26 per cent results in a soil-aggregate mixture with lower strength characteristics than either the aggregate or the binder soil alone.

CHAPTER VII

RECOMMENDATIONS

As laboratory investigation and analysis proceeded, the following questions arose which the author suggests as topics for further research:

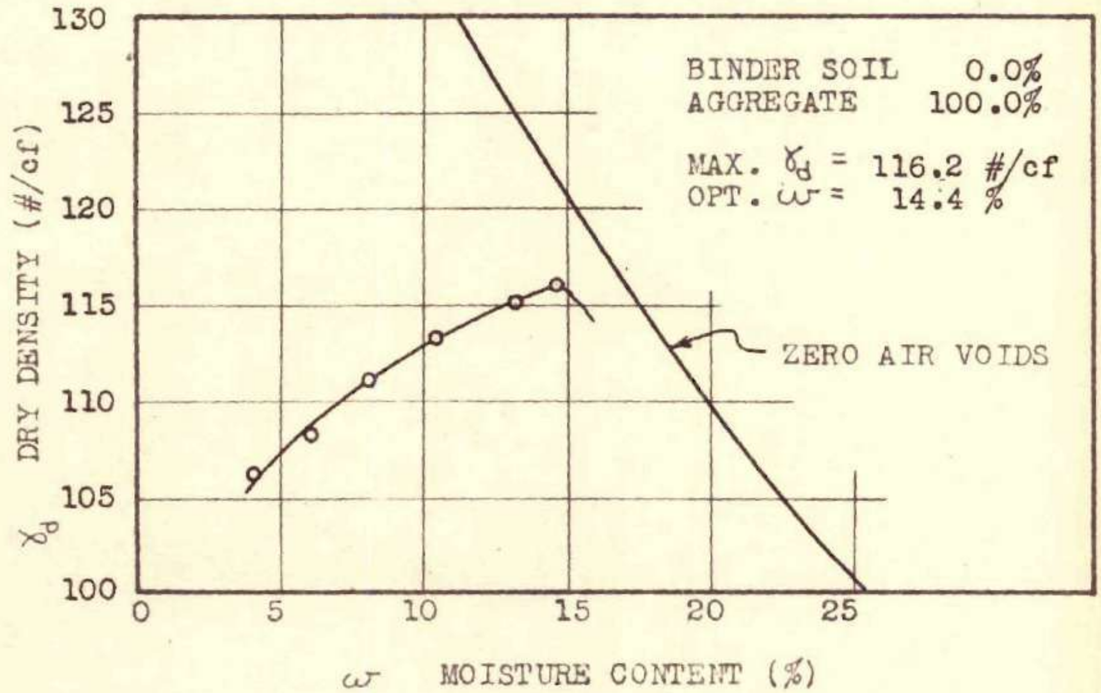
1. Would the use of higher compactive efforts reduce or eliminate the sharp drop in the angle of internal friction experienced in the mixtures with low binder soil contents?
2. Would more precisely defined values of cohesion for mixtures with binder soil contents of from 10 per cent to 30 per cent reveal the presence or effects of partial compaction of the binder soil?
3. Would the swell-shrink characteristics of mixtures containing from 10 per cent to 30 per cent binder soil reveal the presence or effects of partial compaction of the binder soil?
4. Are the variations in strength for increasing binder soil contents the same for the saturated condition as have been found for the partially saturated condition?
5. What is the susceptibility of the various mixtures to freezing and thawing?
6. Can adequate design procedures be developed based on either the percentage of voids filled with binder soil, or the percentage of voids filled with material finer than the No. 200 sieve?

A P P E N D I X

TABLE 2

Summary of Test Results and Computed Data

Percentage of Binder Soil in Mixture	Percentage of Particles Smaller Than 0.002 mm.	Maximum Dry Density (#/cf)	Optimum Moisture Content (%)	Angle of Internal Friction ø (degrees)	Cohesion C (kips/sf)	Degree of Saturation (%)	Volume of Voids in Aggregate (cf/cf)	Volume of Voids in Total Mix (cf/cf)	Percentage of Voids in the Aggregate Filled with Binder Soil
0	0	116.2	14.4	45.7	-	85.2	.314	.314	.0
3.6	1.1	119.0	13.7	38.0	0.43	87.9	.323	.297	8.0
7.3	2.2	120.0	12.7	37.5	0.55	83.8	.344	.291	15.0
11.1	3.2	121.4	12.5	37.5	0.79	85.9	.363	.283	22.0
14.8	4.3	122.0	12.4	35.5	0.68	87.1	.386	.278	29.3
18.4	5.3	123.0	12.0	37.5	0.58	86.4	.408	.273	33.1
22.8	6.5	123.6	12.2	34.0	0.86	90.3	.437	.268	38.7
26.2	7.5	123.6	12.1	36.0	0.72	89.6	.461	.268	41.9
33.8	9.1	122.5	12.2	21.0	1.73	87.2	.521	.274	47.4
40.8	11.7	121.4	12.5	24.5	1.80	84.3	.576	.280	51.4
47.6	13.7	119.6	13.5	22.5	1.73	89.3	.630	.290	54.0
100.0	28.8	104.5	19.8	24.0	2.30	88.3	1.000	.375	62.5



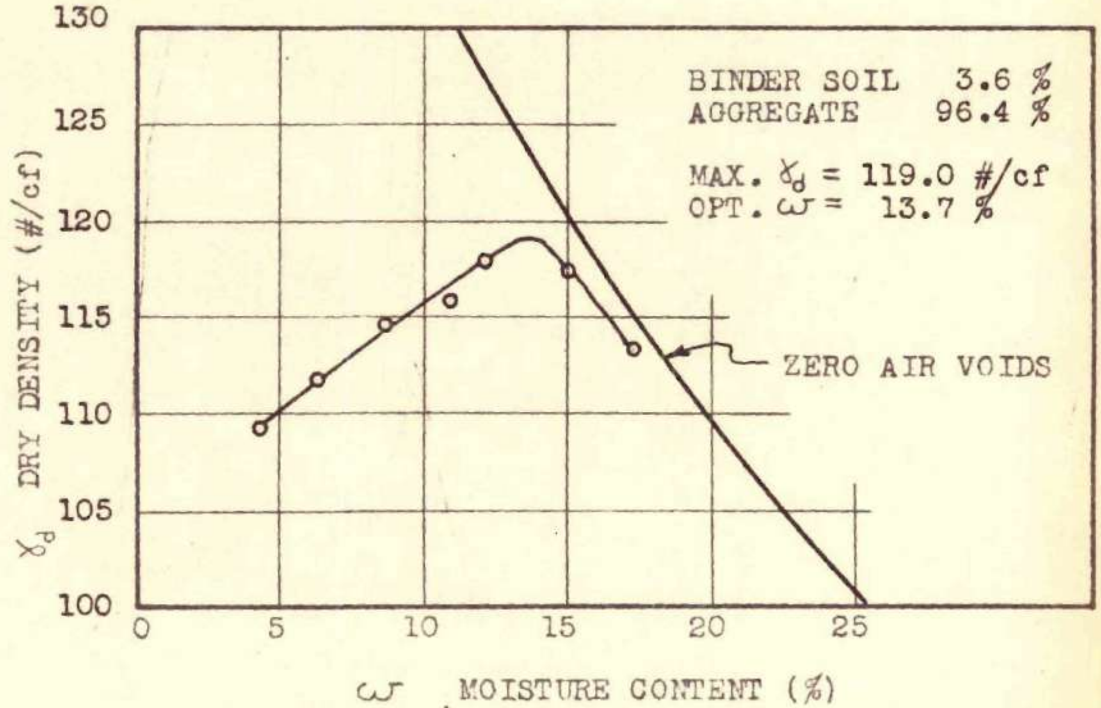


Fig. 20. Moisture-Density Relationship for 0.0% and 3.6% Binder Soil

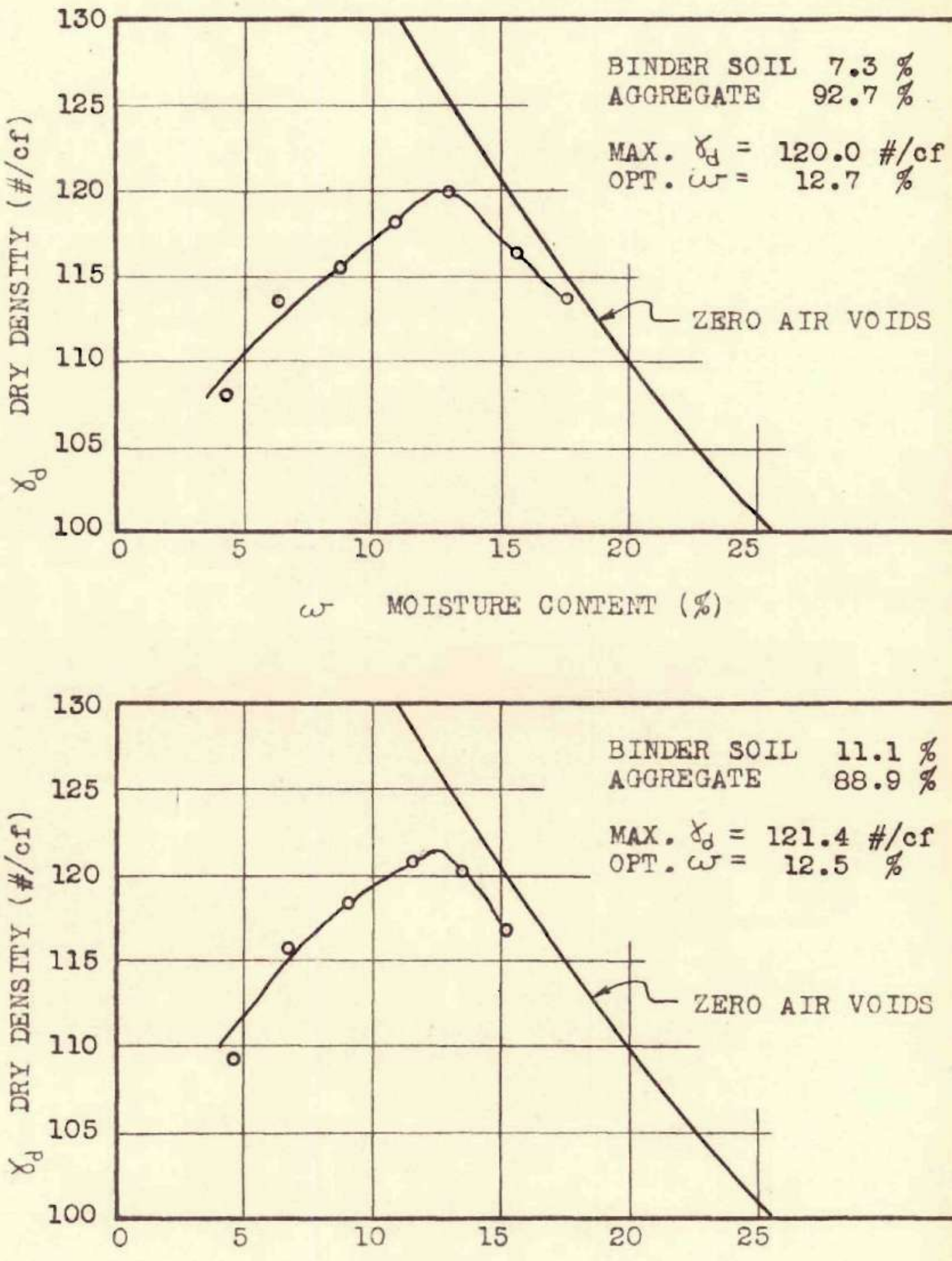
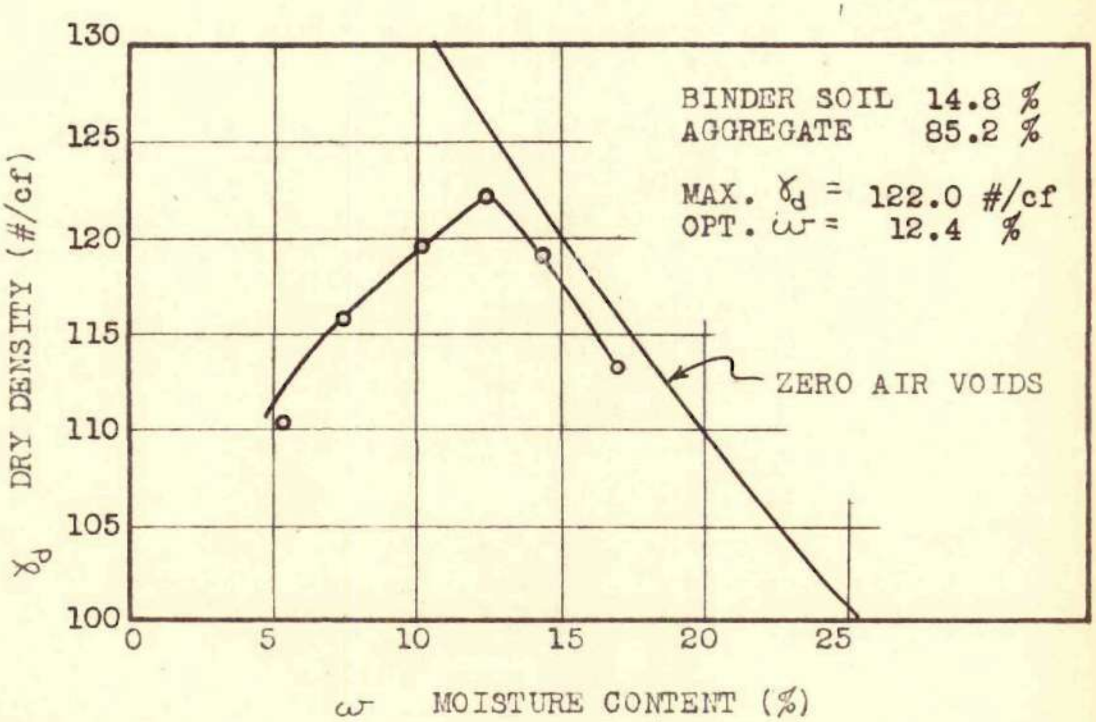


Fig. 21. Moisture-Density Relationship for 7.3% and 11.1% Binder Soil



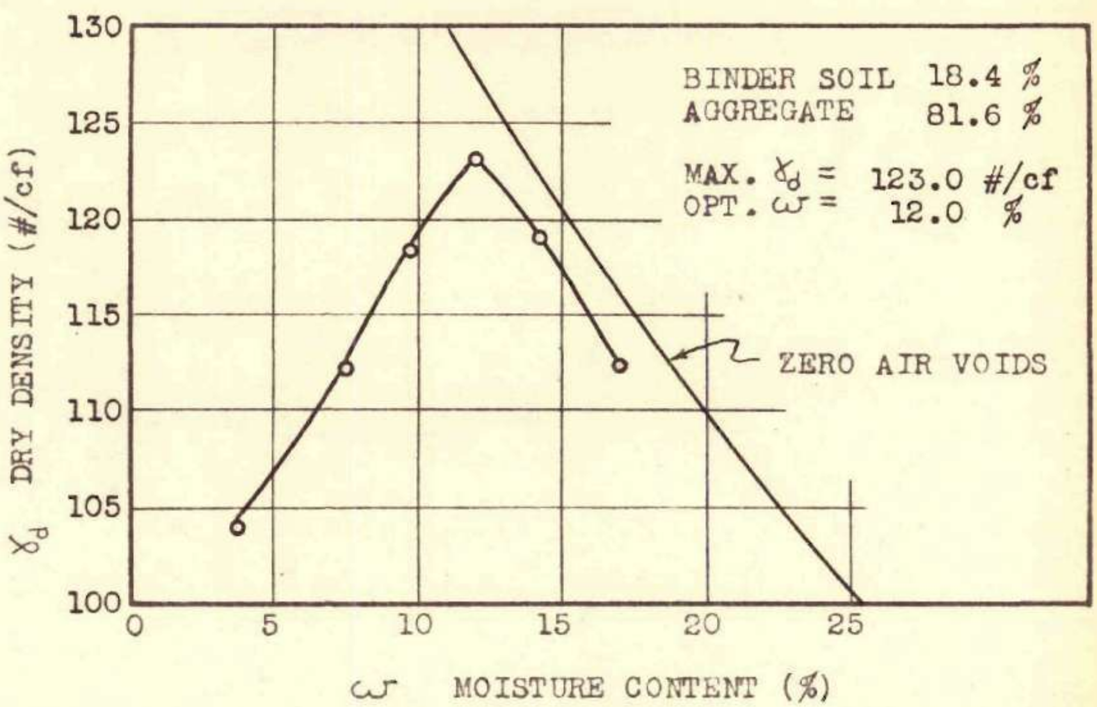
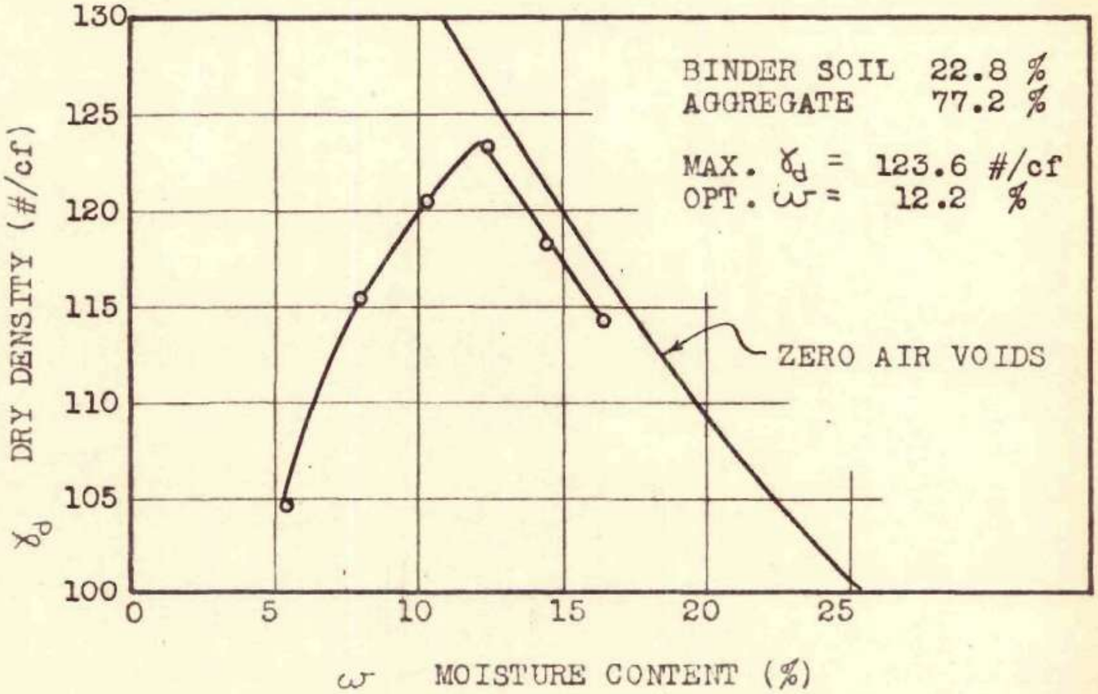


Fig. 22. Moisture-Density Relationship for 14.8% and 18.4% Binder Soil



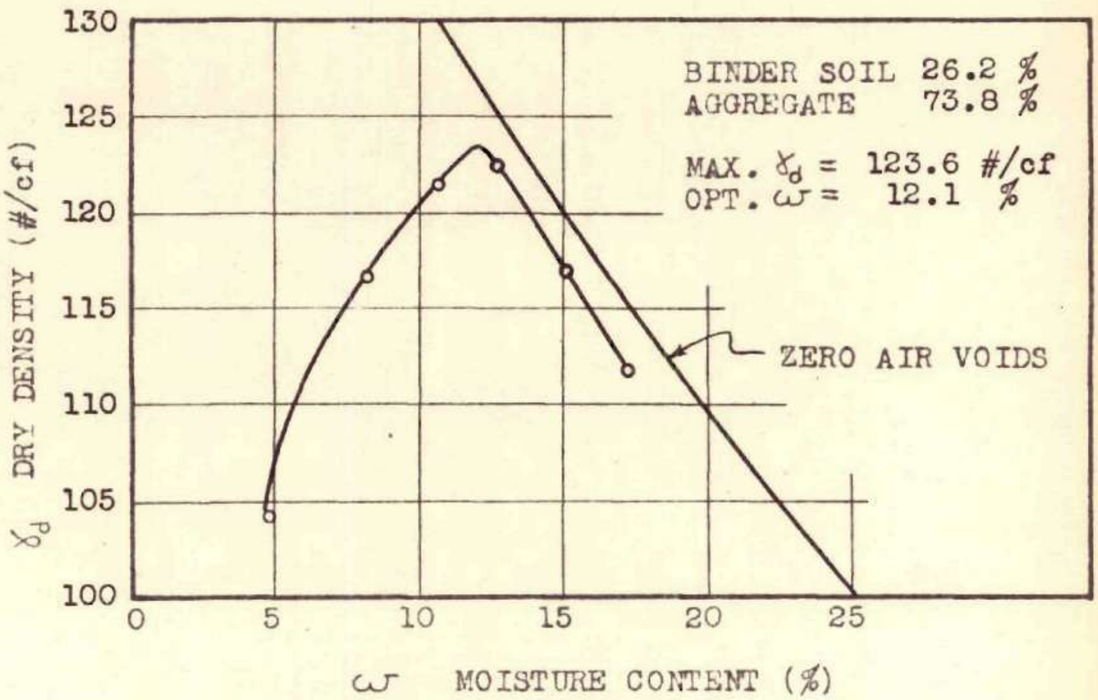
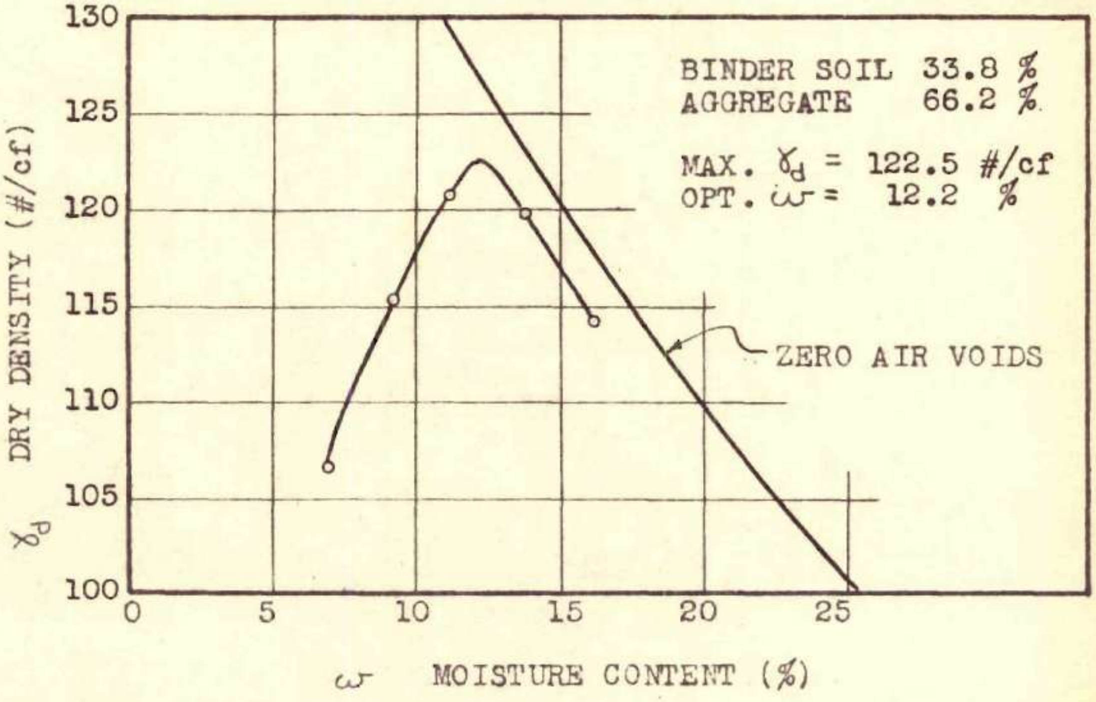


Fig. 23. Moisture-Density Relationship for 22.8% and 26.2% Binder Soil



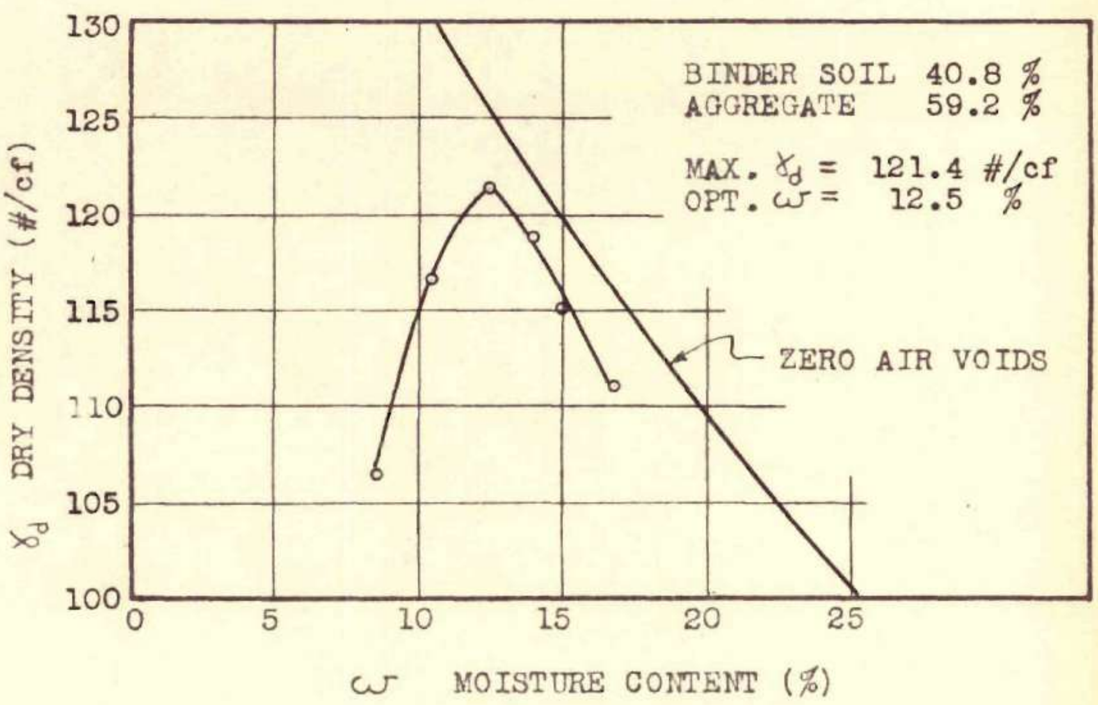
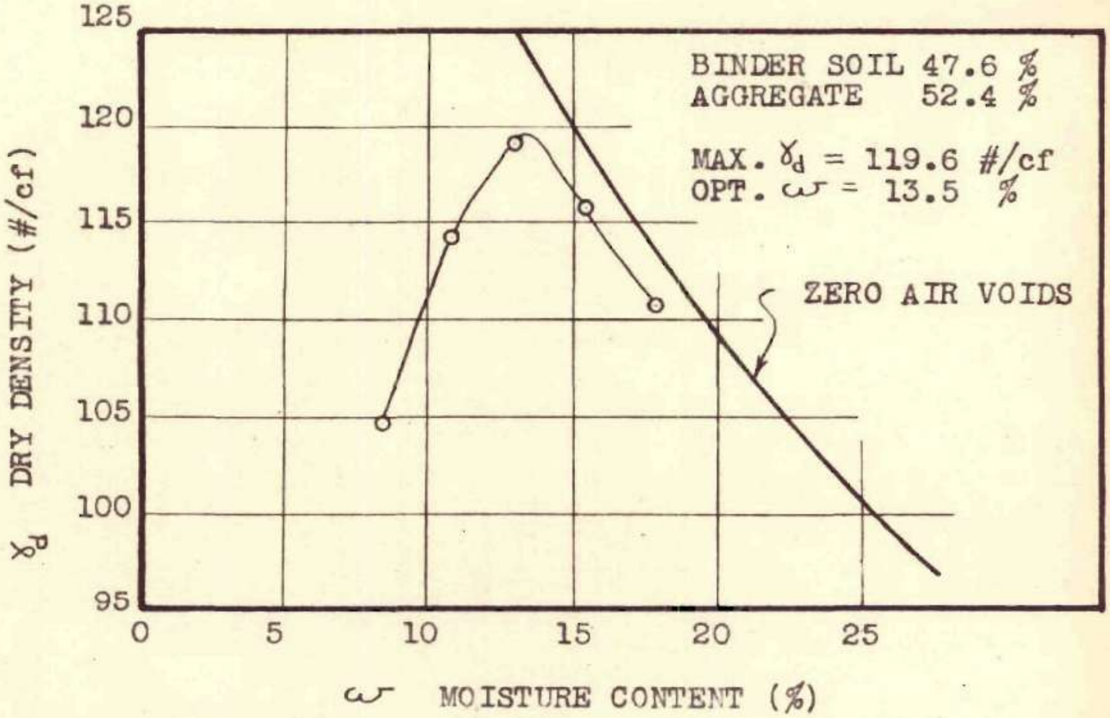


Fig. 24. Moisture-Density Relationship for 33.8% and 40.8% Binder Soil



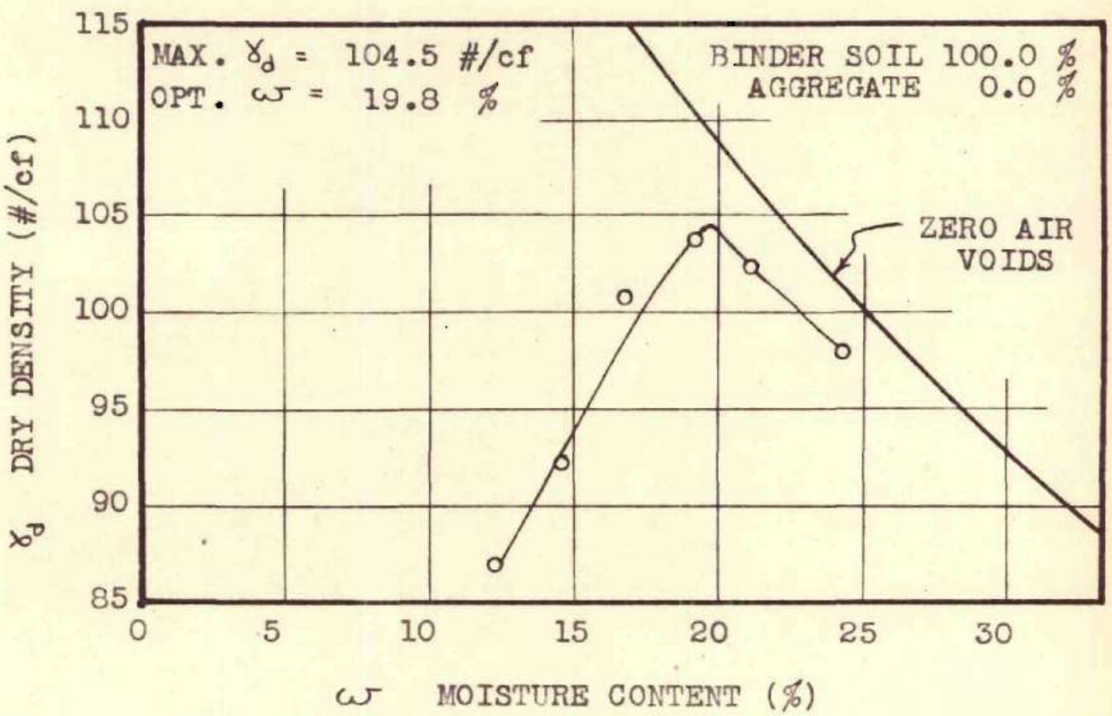
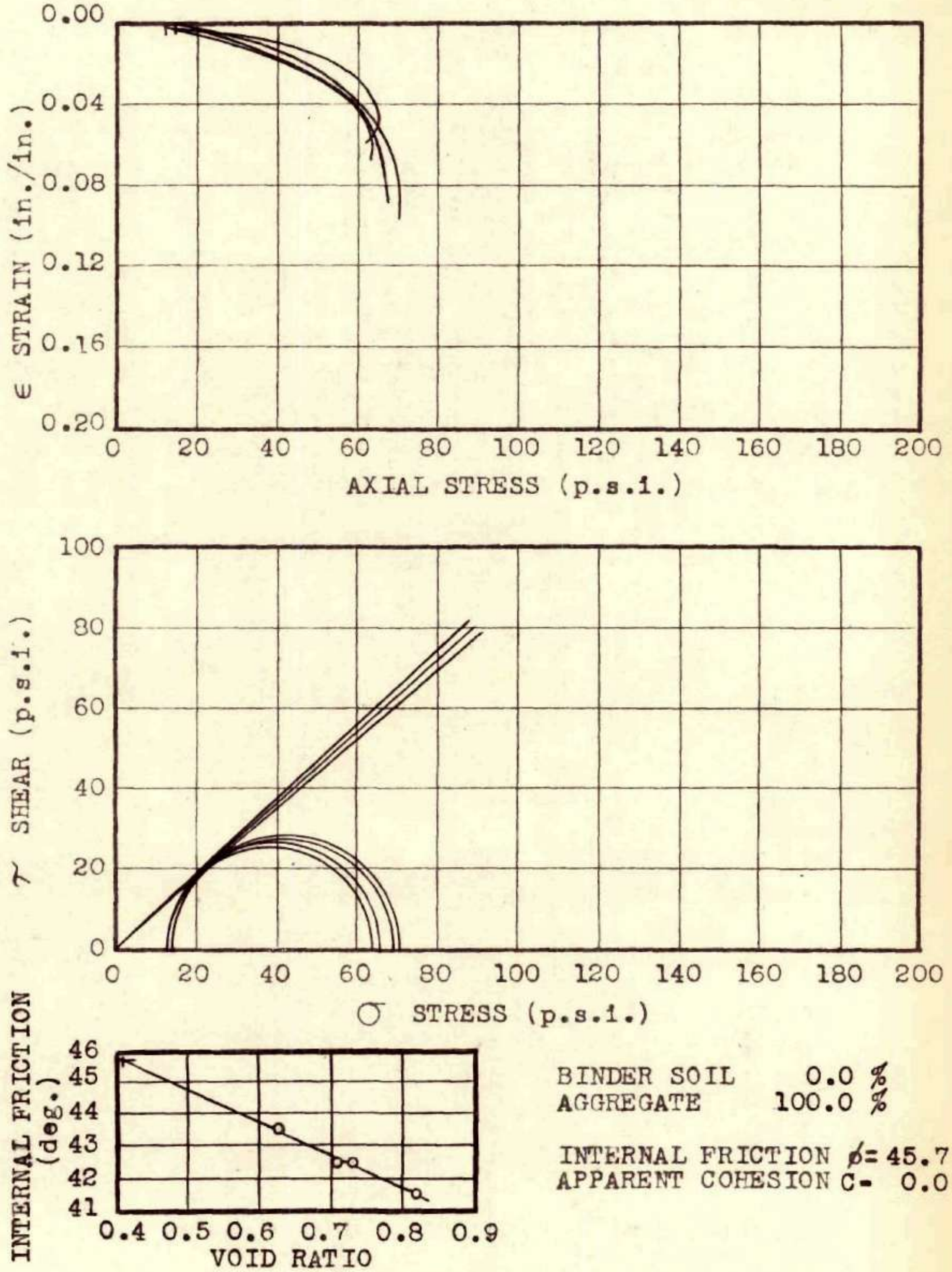


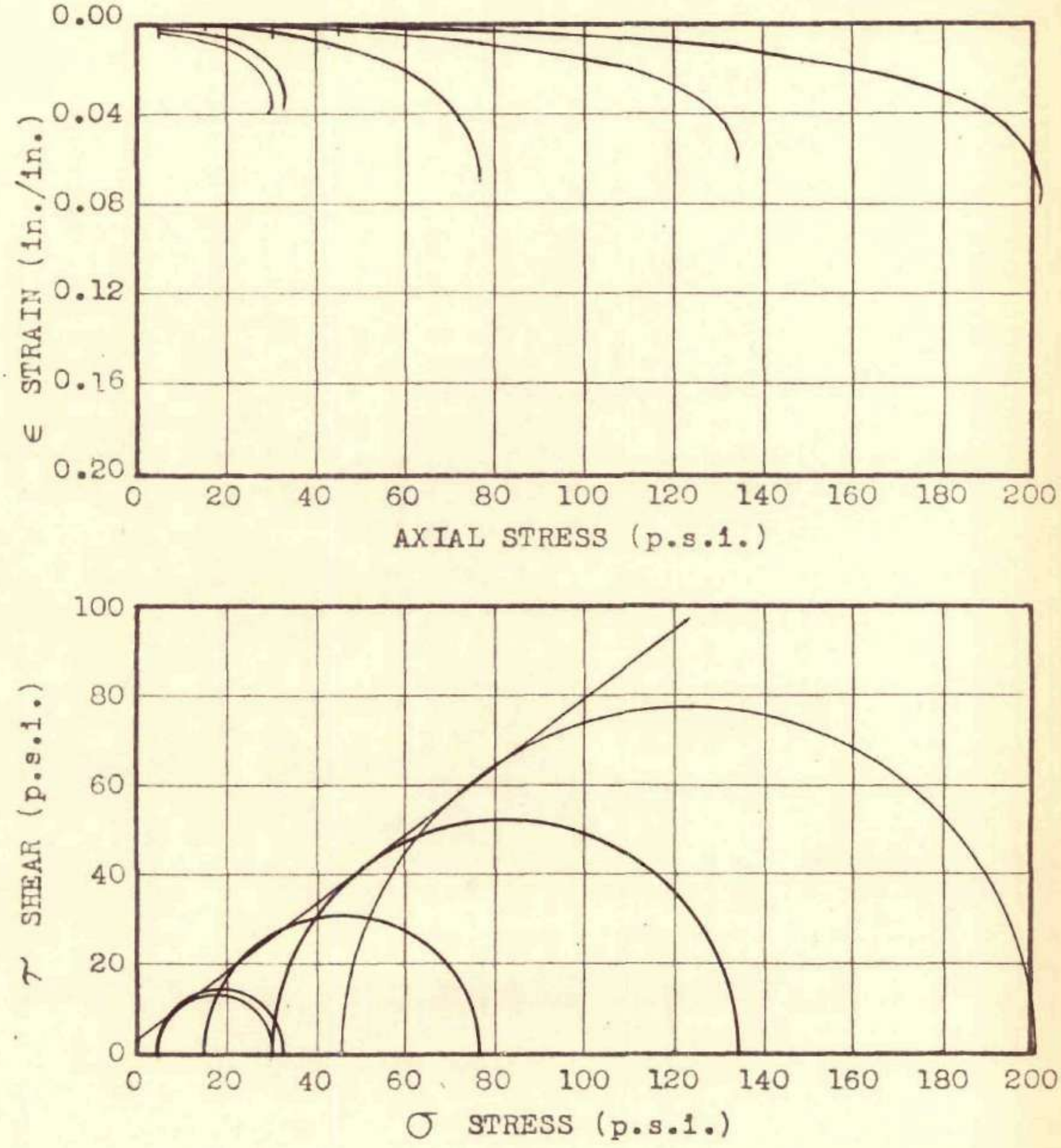
Fig. 25. Moisture-Density Relationship for 47.6% and 100.0% Binder Soil



BINDER SOIL 0.0 %
AGGREGATE 100.0 %

INTERNAL FRICTION ϕ= 45.7
APPARENT COHESION C- 0.0

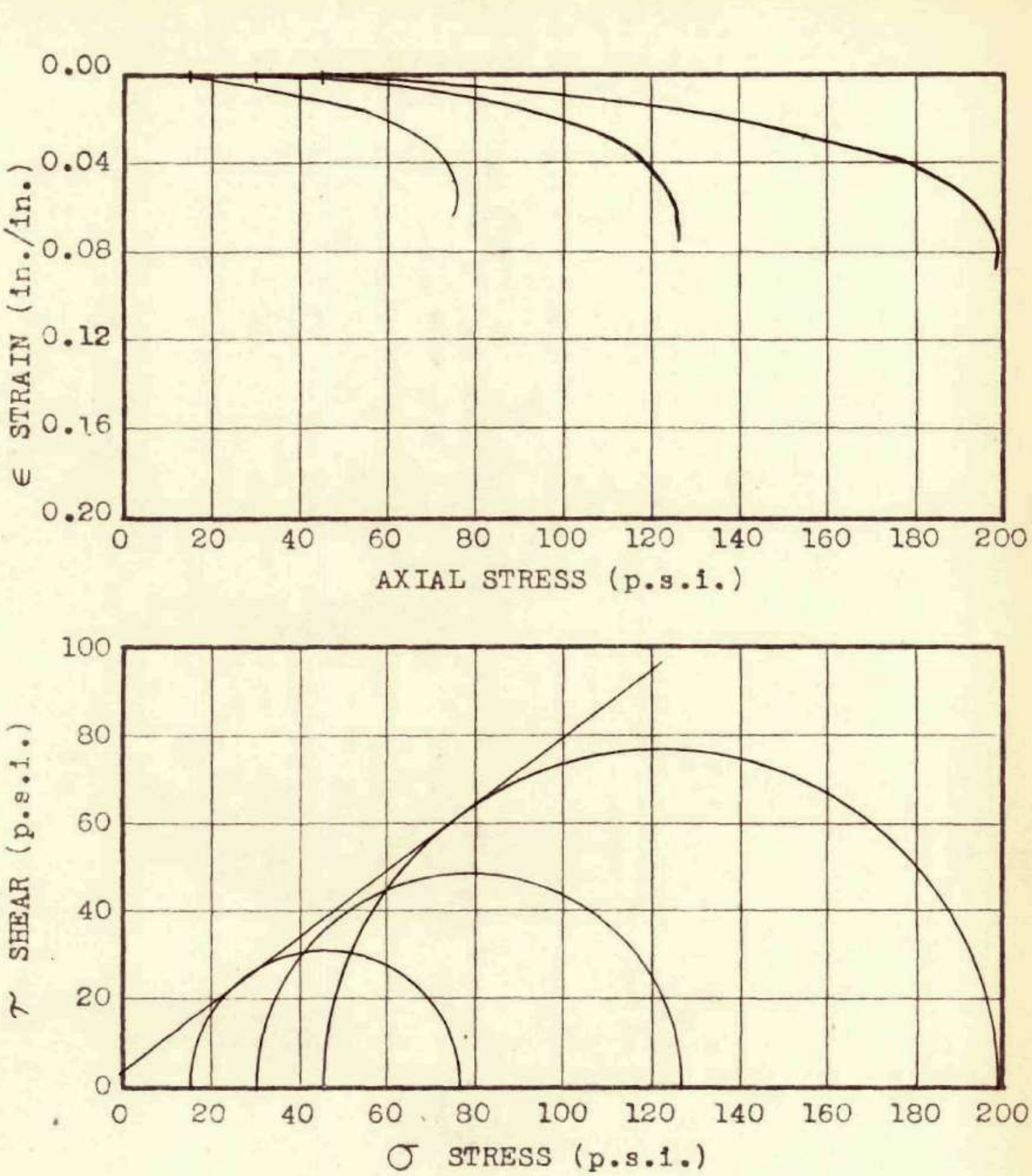
Fig. 26. Strength Characteristics of the Mixture Containing 0.0% Binder Soil



BINDER SOIL 3.6 %
AGGREGATE 96.4 %

INTERNAL FRICTION ϕ=38.0
APPARENT COHESION
C = 3 p.s.i.
= 0.43 k/sf

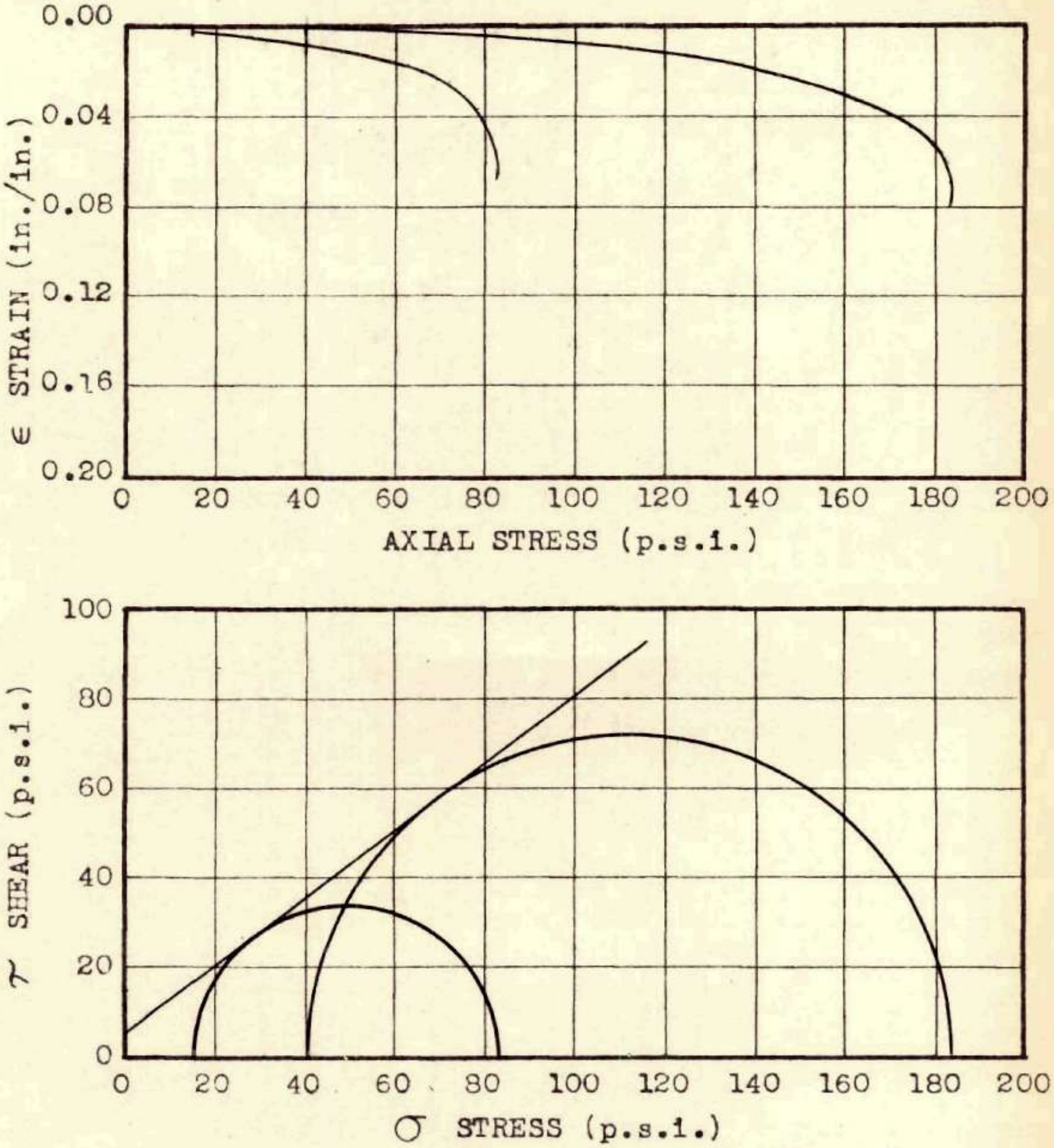
Fig. 27. Strength Characteristics of the Mixture Containing 3.6% Binder Soil



BINDER SOIL 7.3 %
AGGREGATE 92.7 %

INTERNAL FRICTION $\phi = 37.5$
APPARENT COHESION
C = 3.8 p.s.i.
= 0.55 k/sf

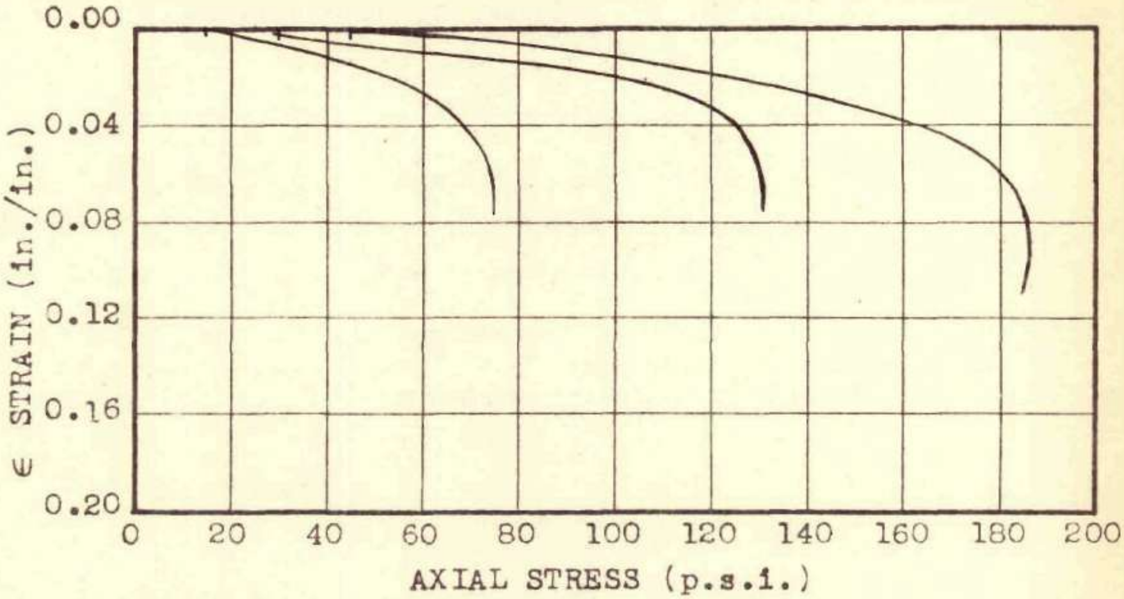
Fig. 28. Strength Characteristics of the Mixture Containing 7.3% Binder Soil

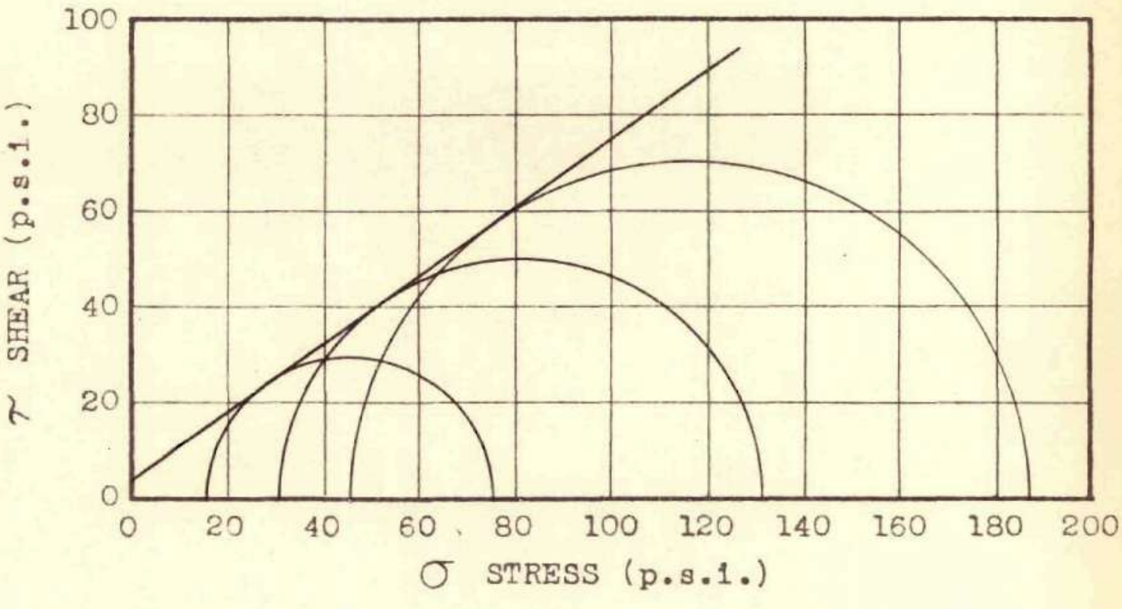


BINDER SOIL 11.1 %
AGGREGATE 88.9 %

INTERNAL FRICTION ϕ=37.5
APPARENT COHESION
C = 5.5 p.s.i.
= 0.79 k/sf

Fig. 29. Strength Characteristics of the Mixture Containing 11.1% Binder Soil

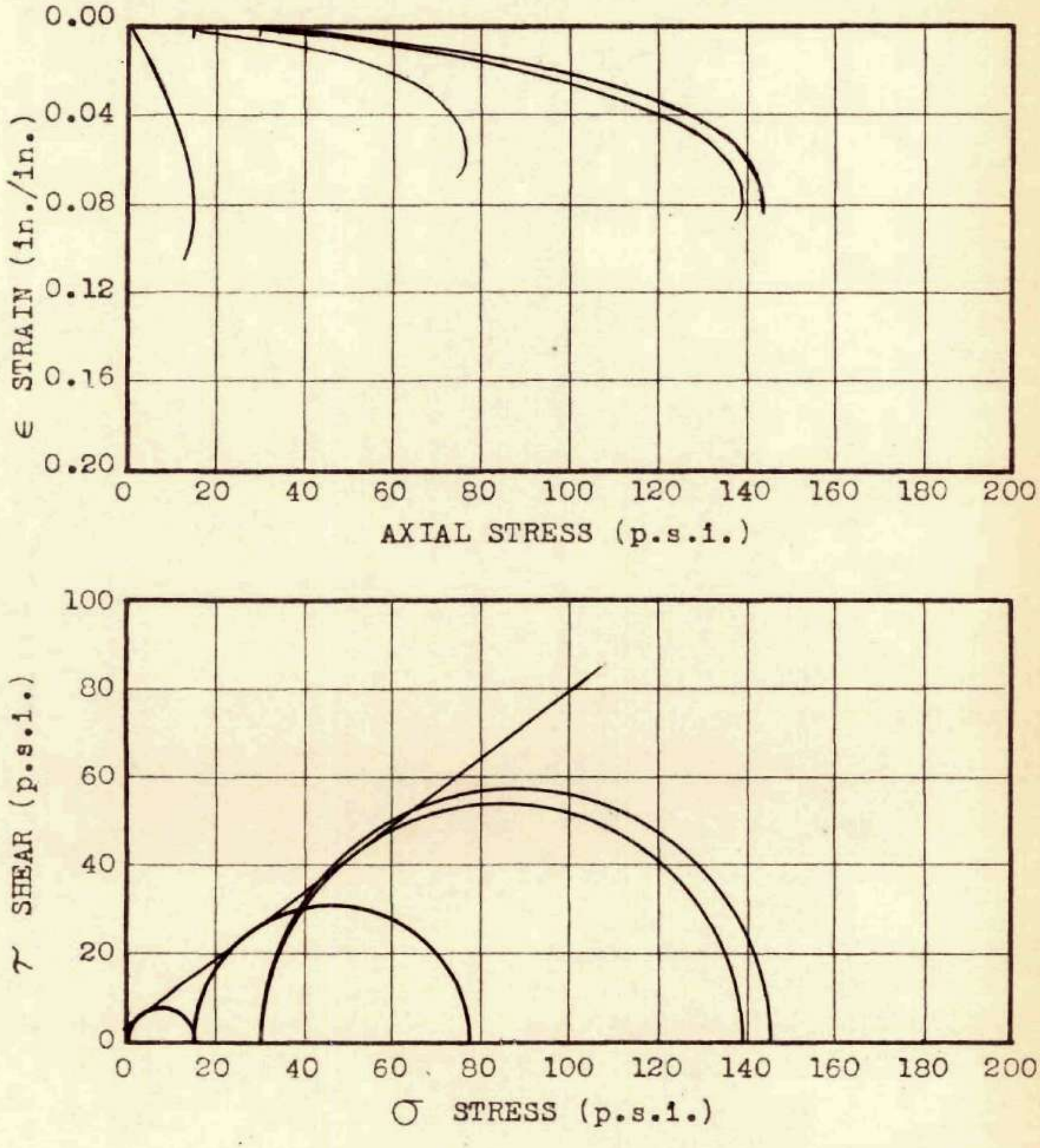




BINDER SOIL 14.8 %
AGGREGATE 85.2 %

INTERNAL FRICTION ø= 35.5
APPARENT COHESION
C = 4.7 p.s.i.
= 0.68 k/sf

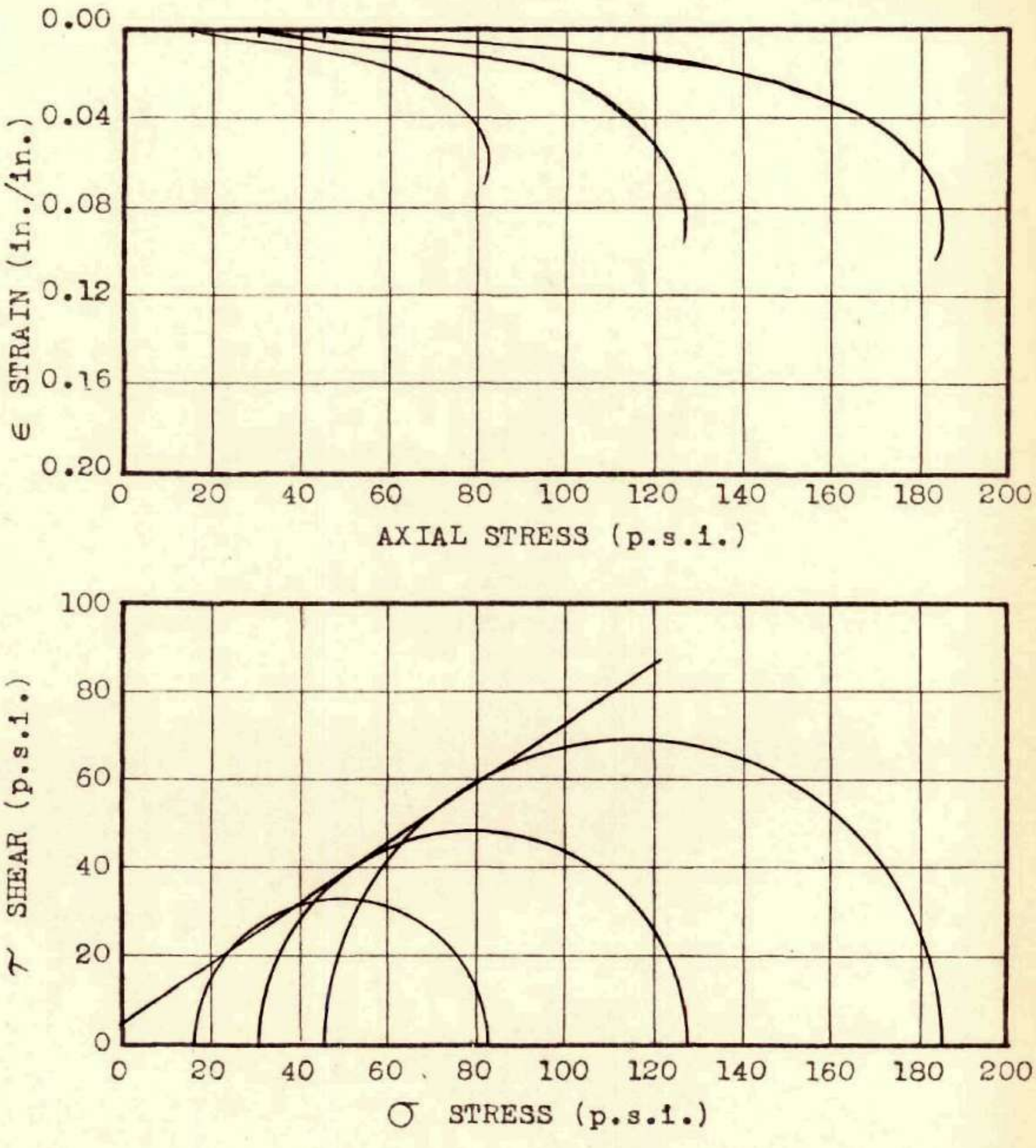
Fig. 30. Strength Characteristics of the Mixture Containing 14.8% Binder Soil



BINDER SOIL 18.4 %
AGGREGATE 81.6 %

INTERNAL FRICTION $\phi = 37.5$
APPARENT COHESION
C = 4.0 p.s.i.
= 0.58 k/sf

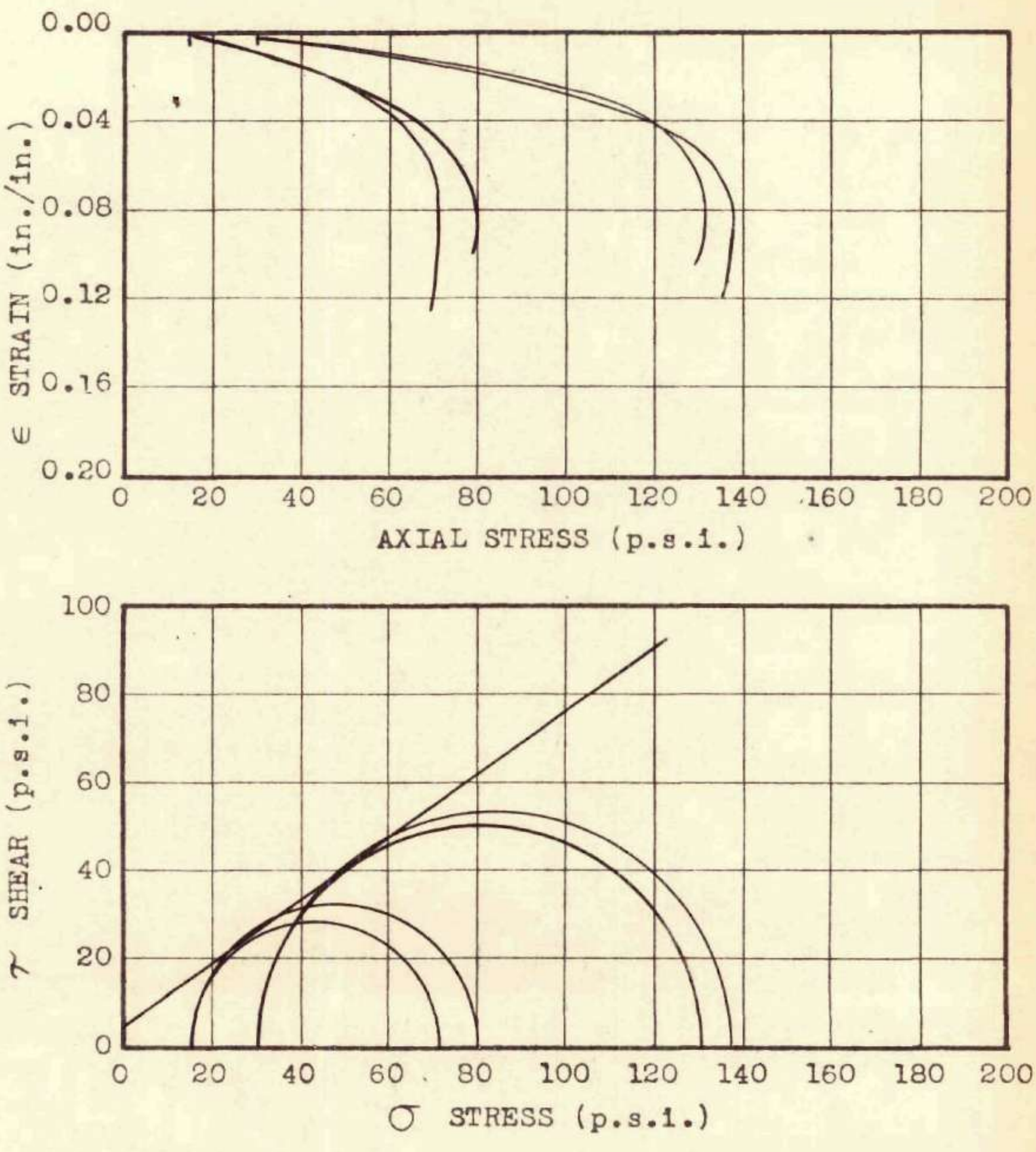
Fig. 31. Strength Characteristics of the Mixture Containing 18.4% Binder Soil



BINDER SOIL 22.8 %
AGGREGATE 77.2 %

INTERNAL FRICTION ϕ=34.0
APPARENT COHESION
C = 6.0 p.s.i.
= 0.86 k/sf

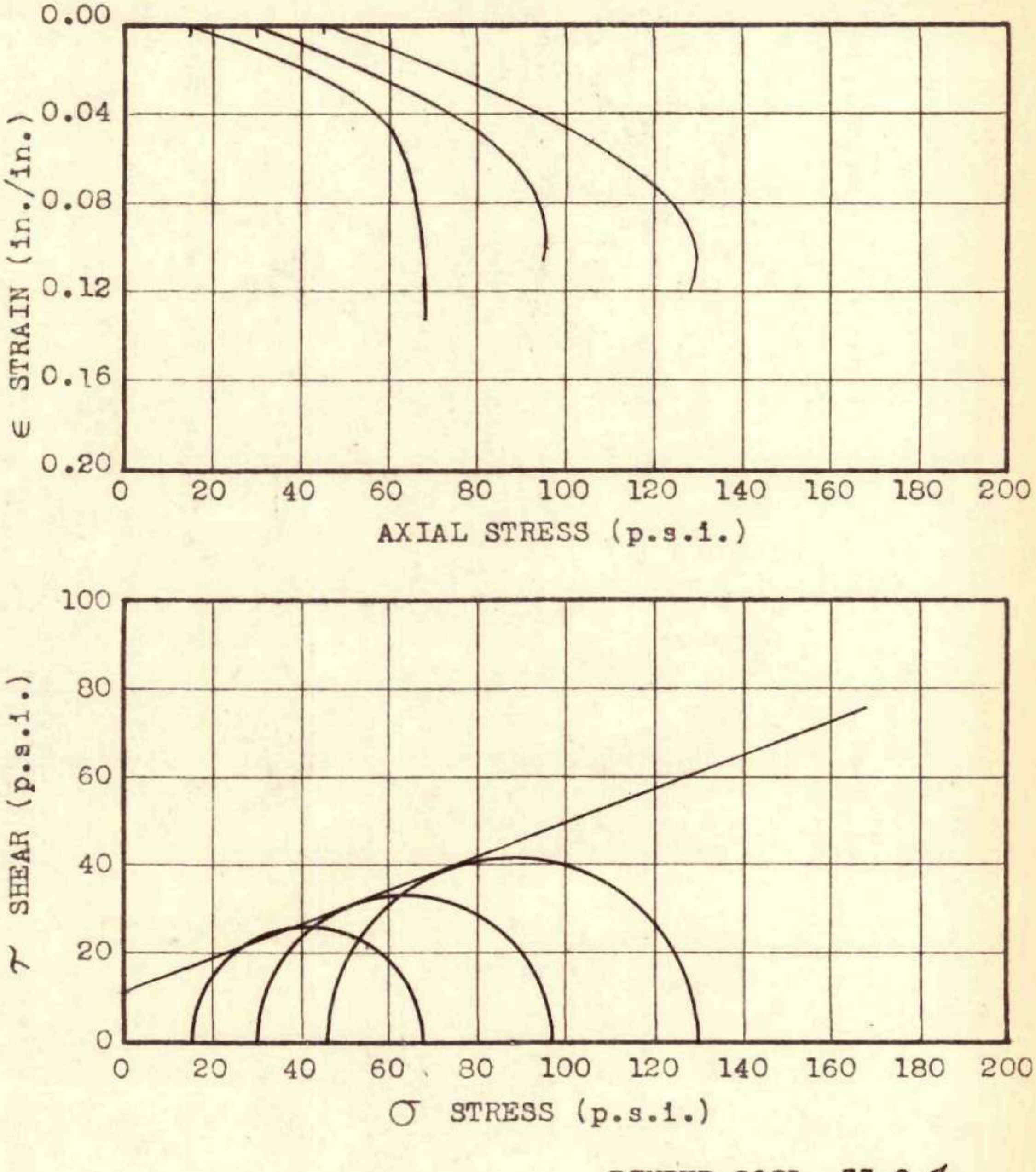
Fig. 32. Strength Characteristics of the Mixture Containing 22.8% Binder Soil



BINDER SOIL 26.2 %
AGGREGATE 73.8 %

INTERNAL FRICTION ϕ= 36.0
APPARENT COHESION
C = 5.0 p.s.i.
= 0.72 k/sf

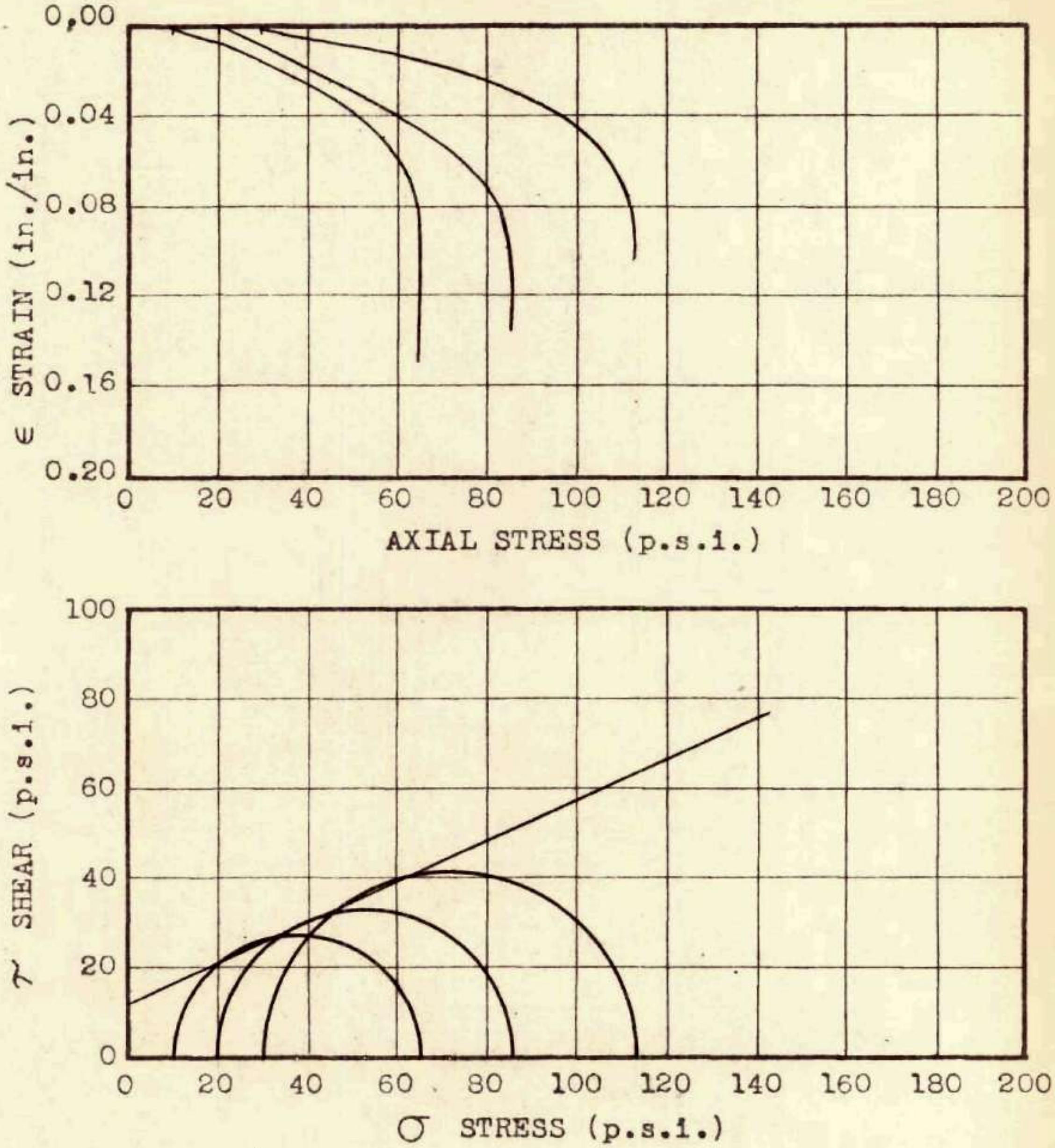
Fig. 33. Strength Characteristics of the Mixture Containing 26.2% Binder Soil



BINDER SOIL 33.8 %
AGGREGATE 66.2 %

INTERNAL FRICTION ϕ=21.0
APPARENT COHESION
C = 12.0 p.s.i.
= 1.73 k/sf

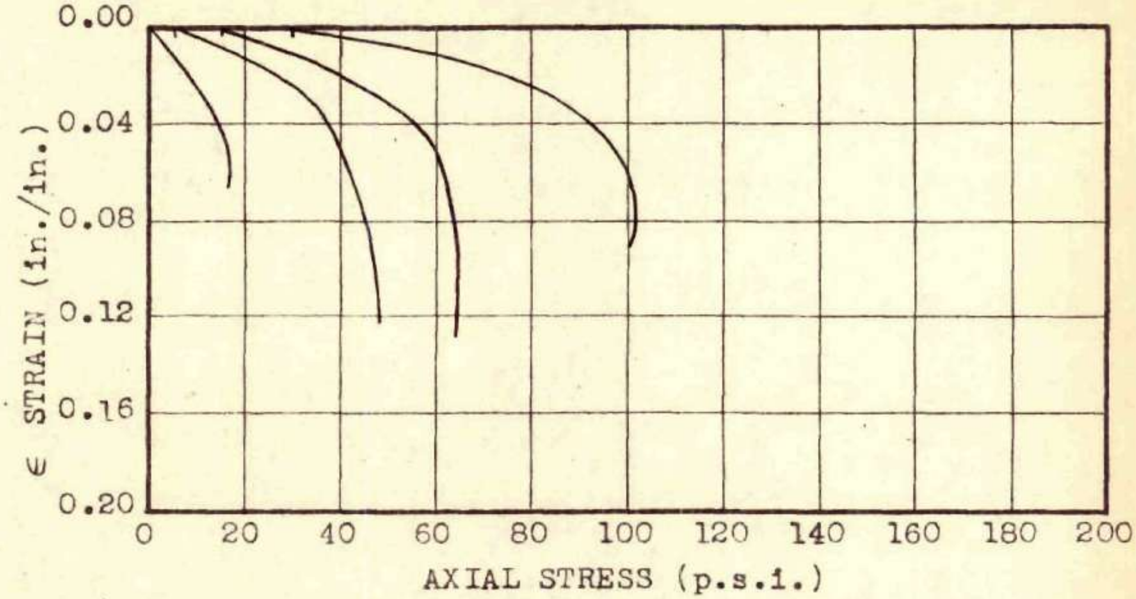
Fig. 34. Strength Characteristics of the Mixture Containing 33.8% Binder Soil

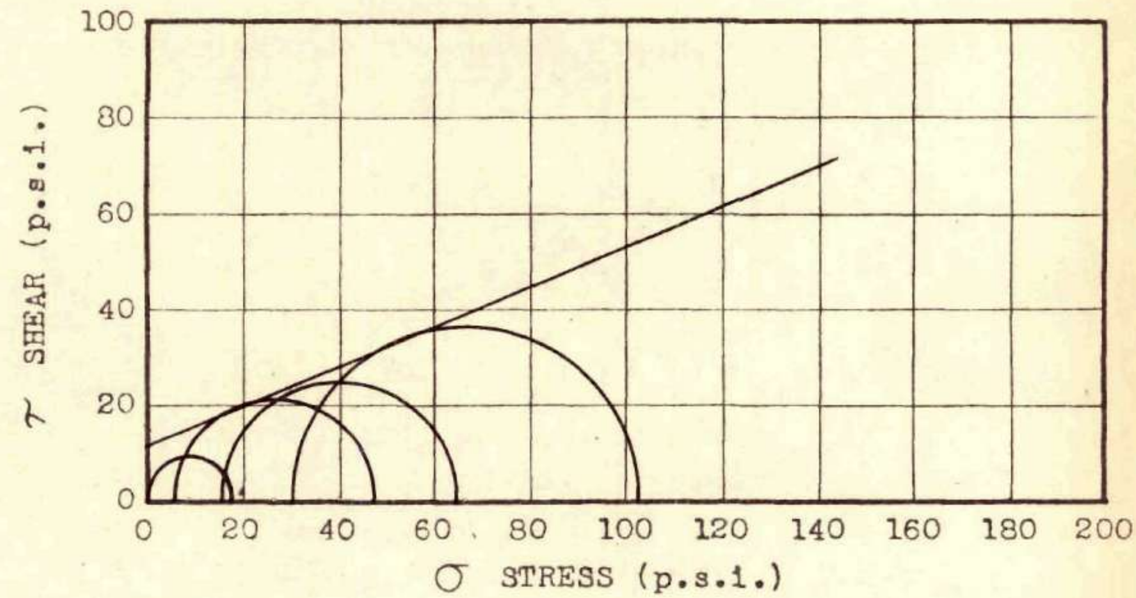


BINDER SOIL 40.8 %
AGGREGATE 59.2 %

INTERNAL FRICTION ϕ= 24.5
APPARENT COHESION
C = 12.5 p.s.i.
= 1.80 k/sf

Fig. 35. Strength Characteristics of the Mixture Containing 40.8% Binder Soil

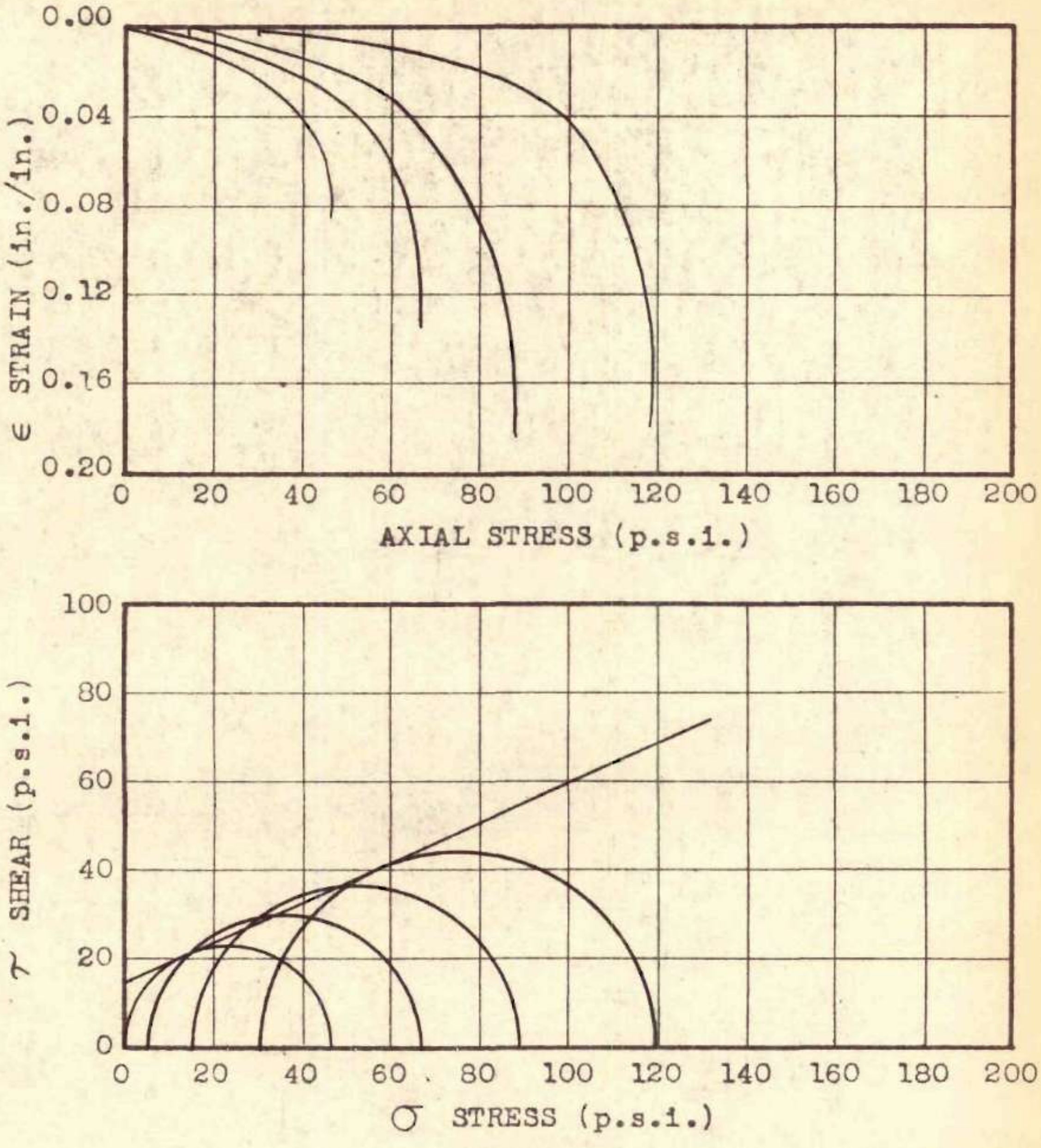




BINDER SOIL 47.6 %
AGGREGATE 52.4 %

INTERNAL FRICTION ϕ=22.5
APPARENT COHESION
C = 12.0 p.s.i.
= 1.73 k/sf

Fig. 36. Strength Characteristics of the Mixture Containing 47.6% Binder Soil



BINDER SOIL 100.0 %
AGGREGATE 0.0 %

INTERNAL FRICTION ϕ= 24.0
APPARENT COHESION
C = 16.0 p.s.i.
= 2.30 k/sf

Fig. 37. Strength Characteristics of the Mixture Containing 100.0% Binder Soil

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